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Consulting Engineering Geologist

June 27, 2011

RBF Consulting
14725 Alton Parkway
Irvine, CA 92618-2027

Attention: Mr. Eddie Torres

Reference: Geology, Soils, Seismicity Report in Support of DWP Specific Plan Amendment EIR, Seal Beach, California
(RBF Consultants JN 10-107353)

Dear Mr. Torres,

In accordance with your request and authorization, the following report presents my EIR-level evaluation of the geological hazards and geotechnical constraints for the proposed residential development on property known as the DWP site in Seal Beach, California. The project site consists of a 10.7-acre site formally utilized by the Los Angeles Department of Water and Power for power plant facilities and operations. As currently planned, the proposed development includes 48 single-family residential lots and interior streets within the eastern one-third of the site. No grading plans depicting proposed grades were available for this study.

As part of this evaluation, a limited subsurface investigation was conducted, which included drilling, logging, and sampling two, 75-foot-deep rotary-wash borings; and 7 Cone Penetrometer Tests (CPTs). In addition, laboratory testing of selected soil samples obtained from the drilling program, and limited geotechnical engineering analyses were performed by my subconsultant, AMEC Geomatrix, to augment this evaluation. Findings from these evaluations are presented herein.

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Based on the results of this study, there are a number of significant geologic hazards and geotechnical constraints to the proposed development, including the following:

- Strong, vibratory ground motion from future earthquakes;
- Liquefaction, lateral spreading, ground lurching, and seismically-induced landsliding of the adjacent levee for the San Gabriel River channel;
- Seismically-induced settlement soil settlement;
- Potential flooding due to tsunami run-up;
- Shallow groundwater;
- Corrosive soils; and
- Sloughing and caving of excavations.

There are no active or potentially active faults within or projecting towards the property.

If you have any questions or require additional information, please call.

We appreciate the opportunity to assist RBF Consulting and the City of Seal Beach. If you have any questions or comments, please call.

Respectfully Submitted,

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1.0 INTRODUCTION

The scope of work performed as part of the geology, soils and seismicity portion of the Seal Beach DWP Site (Site) Environmental Impact Report (EIR) included compilation and review of published geologic and seismic hazards maps, historic black-and white aerial photographs obtained from UC Santa Barbara, civil design drawings of the adjacent San Gabriel River Channel (SGRC) levee, and a geotechnical report for the site prepared by GeoTek, Inc. (GeoTek, 2005). A list of the reports, maps and other relevant data reviewed for this study are presented in the References section at the end of this report.

In addition, a field exploration program was performed by this office that included drilling, logging, and sampling two, 75-foot-deep rotary wash borings, and performing five, 75-foot-deep Cone Penetration Test (CPT) soundings. This was then followed by laboratory testing of selected soil samples and geotechnical engineering analyses associated with seismically-induced liquefaction, lateral spread and landsliding. Details pertaining to the soils encountered and samples obtained are presented on the Boring Logs in Appendix A. Limited laboratory tests were performed to provide a basis for the geotechnical evaluation of the site. The results of the laboratory tests performed on the selected samples are presented in Appendix B.

As with all new construction, requirements for geotechnical and geologic/ seismic hazard studies are provided in Title 24 of the California Code of Regulations.

The results of the EIR-level evaluation for this study as well as pertinent impacts and mitigating measures are provided in the following report.

2.0 EXISTING CONDITIONS

2.1 GEOLOGIC SETTING

The project site is situated within a coastal lowland area referred to as the Alamitos Gap (Gap), a portion of the Orange County Coastal Plain. The creation of the Gap began in Late Pleistocene time (about 60,000 years before present [ybp] and continued until the end of the last glacial period, approximately 15,000 ybp. The combination of a lowered sea level and accelerated stream erosion produced the ancestral San Gabriel River valley, which is, at most, approximately 100 feet deep, and about a mile wide. At the end of the glacial period, the sea level began to rise, and the ancestral river began backfilling the valley with coastal alluvial deposits. Much of what is known about the subsurface conditions and late Pleistocene erosion and subsequent Holocene-age (0 to 11,000 ybp) sediment deposition in the region has been reported by the U.S. Geological Survey (USGS), California Geological Survey (CGS), California Department of Water Resources (DWR), the Orange County Water District (OCWD), and a number of site-specific investigations performed by various consulting firms for local agencies.

The Gap is underlain at shallow depths by Holocene sediments consisting of ancient river and flood plain (i.e. fluvial and alluvial) deposits associated with the San Gabriel River, and near-shore estuarine, delta, and lagoonal (i.e. paralic) deposits. These sediments consist of unconsolidated sand, gravel, silt, and clay. Erosional remnants of what is interpreted by the CGS (2003) to be older, more consolidated paralic deposits of late to middle Pleistocene-age, underlies the man-modified, low-lying coastal bluff that was situated along the coastline between Anaheim Bay and the modern SGRC.

The Site, which is underlain by both Holocene and late to middle Pleistocene paralic deposits, is mantled by approximately 3 feet of artificial fill soils that were imported to the Site following the demolition of the former Los Angeles Gas and Electric Corporation (LAG&E) plant. The main plant existed in the south central portion of the Site along with appurtenant structures within the northern half of the area between 1925 and 1967 (refer to Figure 1- Site and Geologic Map).

According to State of California Division of Oil, Gas, and Geothermal Resources District 1, Map 132 (dated, August 14, 2007), the project site is not situated within an active or historic oil or gas field. The nearest active oil fields are the Seal Beach and Wilmington fields, which are located about one mile north and south of the Site, respectively. The closest active oil wells lie within the Hellman Ranch field that is located about 1.1 miles to the north, and is associated with what is referred to as the Newport-Inglewood Structural Trend. A number of other significant oil fields are located along the Newport-Inglewood Trend, all of which owe their existence largely to the Newport-Inglewood fault zone (NIFZ).

There are no documented mineral deposits or significant paleontological (i.e. fossil) sites known within the Site.

2.2 SITE CONDITIONS

Historically, before 1868, much of the central portion of the Site was represented by the northwestern most tip of a low-lying coastal bluff. The low bluff was bounded on the north and west by Holocene age (0 to 11,000 year old) alluvial/fluvial (i.e. stream lain) deposits, and on the south by both fluvial and near shore marine sediments. During this time, the outlet for the San Gabriel River was located about 2800 feet further up the coast (Kenyon, 1950; Poland, 1959). Between the period between 1868 and 1931, the San Gabriel River had migrated southward and cut a new channel to the ocean with its outlet adjacent to the western margin of the Site, which, in 1925, was partially occupied by LAG&E steam electric generating plant (Kenyon, 1950). Refer to Figure 1- Site Geologic Map, for location of the LAG&E plant.

During the late 1920s, the Los Angeles County Flood Control District (LACFCD) had improved the lower reach of the San Gabriel River by straightening and widening the channel to a point about 4000 feet northerly of the outlet of the SGRC, which was located directly adjacent to the western side of the LAG&E plant. It was not until 1931 that the LACFCD prepared design plans to improve the remaining southern 4000 feet of the San Gabriel River to its outlet with the Pacific Ocean (LACFCD, August 1931). These plans called for deepening and widening of the natural river channel, constructing a rock bulkhead along the majority of the channel section next to the Site, and placing rock facing along a 100-foot-long section of the channel next to the water intake structure for the plant. According to these circa 1931 design plans, there was an LAG&E underground bulkhead along all but about 200 feet of the edge of the channel directly next to the plant. This bulkhead extended to a depth of about (-)30 feet mean sea level (msl) and was bordered on its western side by a 25-foot-wide, 1- to 10-foot-high, sloping section of protective rip-rap. The 1931 design plans also indicate that the 200-foot-long section along the edge of the channel steel sheet piling had been driven by LAG&E to a depth of about (-)22 feet msl. Based on a review of aerial photographs taken on May 17, 1940, it appears that LACFCD's channel improvements had been implemented.

By 1952, the LAG&E site, which was now owned by the Los Angeles Department of Water and Power (LADWP), was occupied not only by the main plant and what appears to be a nearby cooling tower array, but by a row of buildings along the western edge of 1st Street, and two large, above-ground fuel storage tanks in the northwest and northeast corner of the Site. A review of aerial photographs taken on June 20, 1966, depicts similar conditions, except some of the buildings along 1st Street have been removed.

During the late 1950s to early 1960s, the US Army Corp of Engineers (USACE) redesigned and constructed a new levee section along each side of the outlet for the SGRC. According to USACE's as-built plans, dated May 8, 1963 (provided by County of Los Angeles Department of Public Works, Watershed Management Division), the new levee section adjacent to the Site involved increasing the size and extent of former levees constructed by the LACFCD.
A typical profile showing the original levee circa 1931 and this new levee is shown on Figure 1. It is unknown if the earlier LAG&E bulkheads and/or the sheet piling was left in place or removed during the reconstruction.

By 1966, the LAG&E plant was decommissioned, and it was demolished in 1967. A request was made by this office of LADWP to research their records for geotechnical or design information regarding the original design of the plant, or the demolition and restoration of the property, but was not successful. However, during the course of the exploratory CPT work carried out by this firm, two of the CPTs (CPT-2A and 2b) were situated within the footprint of the former plant and met with refusal at 8 feet below ground surface (bgs). Based on the inability of the CPTs to penetrate any further, it is presumed that there may be some portions of the foundation, or other underground structures, still remaining of the former plant.

Currently, the project area is essentially featureless with ground surface elevations ranging from about 10 feet above msl along the western margin where it abuts the levee of the SGRC, to a high of about 18 feet msl in the south-central portion of the area. This subdued topographic high corresponds to the former natural, low-lying bluff that occupied the south central portion of the Site (see Figure 1).

2.3 GEOLOGIC MATERIALS

The geologic materials that underlie the Site include artificial fill soils, deposits associated with the San Gabriel River and near-shore estuarine, delta, and lagoonal (i.e. paralic) deposits of Holocene and mid to late Pleistocene age.

2.3.1 Artificial fill (Geologic Map Symbol afu)

Artificial fill soils reportedly form a 3-foot-thick veneer across the entire Site. In the vicinity of the levee, varying thicknesses of artificial fill occupy the area between the bike trail and the SGRC right-of way, which appears to coincide with an existing chain link. Given the past history of the Site, it would not be surprising to find other areas underlain by significant amounts of fill soils or other construction debris.

Based on observations during drilling of the exploratory borings and results of the CPT surveys, artificial fill soils are loose, dry, porous, and may contain varying amounts of inorganic debris/ trash. Where these "non-engineered" types of soils are encountered, they are highly erodible and expected to be compressible, and therefore subject to consolidation. If not removed and/or replaced with compacted fill beneath proposed buildings, the foundations and/or structural elements could experience moderate to significant distress.

2.3.2 Holocene Age Paralic Estuarine Deposits (Geologic Map Symbol Qpe)

These unconsolidated sediments have been deposited in a near-shore estuarine and delta-like environmental setting, and include sediments deposited by intermittent stream flows and periods of severe flooding during the Holocene (last 11,000 years). Given the proximity to the ocean, these deposits are also intermixed with near shore marine deposits, and may likely contain salt and other evaporates (California Geological Survey, 2010). Based on information obtained from the exploratory borings and CPTs, these sediments consist of layers and lenses of sand, silt, clay, and mixtures thereof, and were encountered to depths of up to about

55 feet bgs. Some of the finer grained silts and clays contain scattered remains of small plant fragments and other organic detritus.

The geotechnical character of these Holocene age deposits and the presence of the SGRC present significant geotechnical constraints for development of the proposed residential development. As discussed further in this report, these soils are subject to liquefaction, lateral spread, seismically-induced landsliding, and are corrosive to ferrous metals.

2.3.3 Late to Middle Pleistocene Age Paralic Deposits (Geologic Map Symbol Qopa)

Based on geologic mapping by the CGS (2003) and Polland (1959), these older, soil-like deposits form part of a relatively thick, blanket-like deposit that underlies the nearby Landing Hill topographic high, a portion of the Naval Weapons Center, and the former low-lying coastal bluff that occupied the areas between Anaheim Bay and the San Gabriel River. Much of the former LAG&E plant was situated atop the southeasternmost exposure of these sediments. These sediments were encountered in several exploratory borings and CPTs within the Site, including the USACE's 1960 boring TH-69, GeoTek's exploratory boring B-3, and the recent CPT- 2 and CPT-3 that were performed for this study. The approximate subaerial extent of these sediments is depicted on Figure 1; however, additional subsurface studies would be necessary to more accurately define their limits.

Given the relatively dense nature of these deposits, there are no significant constraints associated with liquefaction or lateral spread. However, where these deposits lay astride the SGRC the likelihood for seismically-induced landsliding cannot be precluded at this time.

2.4 Groundwater

Shallow groundwater beneath the Site is saline due to its interconnection with the Pacific Ocean; hence, it is considered a non potable water source. Based on historic and recent groundwater level data, the elevation of the groundwater table beneath the Site varies from about 1 to 5 feet above msl, and mainly is a function of tidal influence from the Pacific Ocean and the water level in the nearby SGRC.

There is no evidence of past or present groundwater use in the project area. No evidence of springs or seeps has been noted within or adjacent to the Site.

2.5 Mineral Resources

There are no economic metallic or non-metallic ore deposits within or in the vicinity of the project area. However, the active Seal Beach and Wilmington oil fields are located approximately one mile north and south of the Site, respectively. According to the State of California Division of Oil, Gas, and Geothermal Resources District 1, Map 132 (August 14, 2007), the closest existing or abandoned oil wells are located within approximately one-half mile of the Site.

3.0 GEOLOGIC HAZARDS AND GEOTECHNICAL CONSTRAINTS

3.1 General

The general nature of paralic deposits, together with shallow groundwater conditions, and the likelihood of moderate to strong earthquake ground motions from future earthquakes in the region, creates significant impacts on the proposed development. The resulting geologic hazards and geotechnical constraints to the proposed development include the following:

- Seismically-induced moderate to strong ground shaking;
- Liquefaction, lateral spreading, and tsunami run-up;
- Seismically-induced landsliding involving the levee next to the SGRC; and
- Seismically-induced settlement of native Holocene age sediments and artificial fill soils that blanket the Site, as well as thicker accumulations of fill soils that lie between the eastern edge of the levee and the SGRC right-of-way.

As currently proposed, all but approximately 7 of the 48 residential lots lie within an area susceptible to liquefaction and lateral spread. Moreover, each of the 13 lots directly adjacent to the levee along the eastern side of the San Gabriel River is susceptible to seismically-induced landsliding.

Although the project area is located within a highly seismically active portion of the state, there are no documented active or potentially active faults transecting or projecting towards the Site. Moreover, there are no documented landslides within or adjacent to the project area.

Non seismic-related geologic hazards at the Site include the presence of corrosive soils and soils subject to sloughing and caving during excavation. There is no current evidence that suggests the presence of soils containing collapsible, organic peat deposits, or expansive soils on the Site.

3.2 FAULTING AND SEISMICITY

The project area is situated within a highly seismically active area of Southern California referred to as the Los Angeles Basin. Hazards associated with earthquakes include primary seismic hazards, such as ground shaking and surface rupture, and secondary seismic hazards, such as liquefaction, seismically-induced settlement, landsliding, tsunamis, and seiches.

In accordance with the CGS, a fault is defined as a fracture in the crust of the earth along which rocks on one side have moved relative to those on the other side. Most faults are the result of repeated displacements over a long period of time. An inactive fault is a fault that has not experienced earthquake activity within the last three million years. In comparison, an active fault is one that has experienced earthquake activity in the past 11,000 years. A fault that has moved within the last two to three million years, but has not been proven by direct evidence to have moved within the last 11,000 years, is considered potentially active. No active or potentially active faults are located within or project towards the project area.

The Alquist-Priolo Act of 1972 (now the Alquist-Priolo Earthquake Fault Zoning Act, Public Resources Code 2621-2624, Division 2 Chapter 7.5) regulates development near active faults in order to mitigate the hazard of surface fault-rupture. Under the Act, the State Geologist is required to delineate "special study zones" along known active faults in California. The Act also requires that, prior to approval of a project, a geologic study be conducted to define and delineate any hazards from surface rupture. A geologist registered by the State of California, within or retained by the lead agency for the project, must prepare this geologic report.
A 50-foot setback from any known trace of an active fault is required. The project area is not currently known to be located within an Alquist-Priolo Earthquake Fault Zone, according to the CGS. The closest Earthquake Fault Zone to the Site is the Seal Beach segment of the NIFZ located about one mile to the north.

Ground shaking accompanying earthquakes on nearby faults can be expected to be felt within the site. However, the intensity of ground shaking would depend upon the magnitude of the earthquake, the distance to the epicenter, and the geology of the area between the epicenter and the property. The Modified Mercalli Intensity (MMI) scale was developed in 1931 and measures the intensity of an earthquake's effects in a given locality, and is perhaps much more meaningful to the layman because it is based on actual observations of earthquake effects at specific places. On the MMI scale, values range from I to XII. The most commonly used adaptation covers the range of intensity from the conditions of: "I "- not felt except by very few, favorably situated, to "XII" – damage total, lines of sight disturbed, and objects thrown into the air." While an earthquake has only one magnitude, it can have many intensities, which decrease with distance from the epicenter. In the case of the 1994 Northridge earthquake, the Santa Clarita/ Newhall area experienced MMIs between VII and VIII (Dewey, et. al., 1995). Ground motions, on the other hand, are often measured in percentage of gravity (percent g), where g = 32 feet per second per second (980 cm/sec^2) on the earth.

3.2.1 Strong Seismically-Induced Ground Motion

The following design parameters have been developed based on criteria presented in, and for use with the 2010 California Building Code (CBC), Chapter 16, Section 1613. The USGS website (http://earthquake.usgs.gov/hazards/designmaps) was utilized to establish seismic parameters. The seismic conditions and parameters are summarized below:

Soil Class	D/F
S_{DS}	1.147
S_{D1}	0.659

De-aggregation of the earthquake magnitudes utilizing the USGS Probabilistic Seismic Hazard Assessment (PSHA) 2008 model indicates a model earthquake magnitude of 7.03. According to the 2010 CBC, Section 1803.5.12, peak ground accelerations are permitted to be estimated as 2/5*S_{DS}, where S_{DS} is estimated in accordance with Section 1613.5.4 of 2010 CBC. Based on 2010 CBC, the level of ground motion at the site is approximately 0.46 g. This value of 0.46g is in agreement with the September 2005 PSHA performed by GeoTek for the proposed residential development at the project site.

However, given the changes to the latest CBC, as well as the nature of the proposed project, the use of more stringent earthquake ground motions (i.e. 2% chance of exceedance in 50 years) should be evaluated as part of the future site-specific geotechnical investigation for the project.

A listing of active faults considered capable of producing strong ground motion at the site, their closest distances to the property, and the maximum expected earthquake along each fault is presented in Table 1. Also presented are generalized evaluations of maximum ground shaking at the project site for the maximum earthquakes, and generalized predictions of the likelihood of such events occurring.

TABLE 1

SUMMARY OF ACTIVE FAULTS AND GENERALIZED EARTHQUAKE INFORMATION

Name	Miles from Site	Maximum Magnitude (M)	Expected Level of Ground Shaking	Likelihood
Newport-Inglewood (Seal Beach)	1.1	7.1	High	High
Palos Verdes (Offshore)	3.0	7.3	High	High
Puente Hills Blind Thrust*	4.5	7.1	High	High
San Joaquin Hills Blind Thrust*	8.0	6.6	High	Moderate
Whittier-Elsinore	16.0	6.8	Moderate	High
Santa Monica	27.5	6.6	Moderate	Moderate
Malibu Coast	29.5	6.7	Moderate	Moderate
Hollywood-Raymond	25.7	6.4 to 6.5	Moderate	High
Sierra Madre-Cucamonga	28.5	6.9 to 7.2	Moderate	High
San Jacinto (Anza)	47.5	7.2	Moderate	High
San Andreas (Mojave)	47.8	7.5	Moderate	High
Santa Susana	60	6.7	Low	Moderate

* These faults are termed "blind thrust faults" because they have no surface exposure. The closest distance from the Site is based on a projection of the rupture area along the subsurface trace of the fault.

The greatest amount of ground shaking at the site would be expected to accompany large earthquakes on the Newport-Inglewood and Palos Verdes faults, and the Puente Hills and San Joaquin Hills Blind Thrust faults. Earthquake magnitudes in the range of M6.5 to M7.3 could produce MMI in the range of VIII to XI within the property.

Significant secondary earthquake hazards include ground deformation associated with liquefaction, lurching, lateral spreading, seismically-induced settlement, earthquake-induced landsliding, and tsunamis.

3.2.2 Liquefaction

Seismic ground shaking of relatively loose, granular soils that are saturated or submerged can cause the soils to liquefy and temporarily behave as a dense fluid. Liquefaction is caused by a sudden temporary increase in pore water pressure due to seismic densification or other displacement of submerged granular soils. Liquefaction more often occurs in earthquake-prone areas underlain by young (i.e. Holocene age) alluvium where the groundwater table is higher than 50 feet bgs. The CGS has designated certain areas within California as potential liquefaction hazard zones. The project site is designated as being within a zone having the potential for earthquake-induced liquefaction.

According to preliminary geotechnical evaluation by GeoTek for the proposed residential development at the site, the most significant geotechnical considerations that will warrant mitigation are potential for earthquake-induced soil settlement, the presence of relatively loose fill materials, and a relatively shallow groundwater level. Limited standard penetration test (SPT) data by GeoTek indicate that the sand and silt layers encountered in their borings B-1 and B-4 (see Figure 1) at various depths between the existing ground surface and 56 feet bgs are highly susceptible to liquefaction during strong ground motion from nearby seismic sources. The extent of the potentially liquefiable layers provided in the GeoTek report is in good accordance with our findings from our subsurface investigation. Limited SPT and CPT data by us show that the sand and silt layers at various depths between the existing ground surface and 76 feet bgs are susceptible to liquefaction.

The California Geological Survey (CGS) has designated certain areas within California as potential liquefaction hazard zones. As shown on the State of California's Seismic Hazard

Zone Map for the Seal Beach 7.5' Quadrangle, the project site lies within an area of high liquefaction potential. This assessment is further validated by the results of the subsurface geotechnical studies performed GeoTek and by this office.

As part of this updated review, we have revisited the topic of liquefaction potential at the site by reviewing the most current ground motion information provided by the USGS (2008), and the current design guidelines provided in the 2010 CBC, which makes reference to ASCE 7-05 (ASCE, 2006). The earthquake parameters used in GeoTek's study was based on the Maximum Credible Earthquake (MCE) on the NIFZ with a magnitude (M) of M6.9, and a peak ground acceleration (PGA) of 0.58g. GeoTek scaled this ground acceleration with respect to an M 7.5 earthquake in order to apply the scaled peak ground acceleration to their liquefaction susceptibility analyses. A PGA of 0.47 g was utilized in their liquefaction evaluation. The current seismic design guidelines as presented in ASCE 7-05 (ASCE, 2006) requires the use of PGA from the MCE earthquake with a 2% probability of exceedance in 50 years as the design ground motion for the evaluation of triggering liquefaction. However, 2010 CBC permits estimation of PGA as $2/5^*S_{DS}$, where S_{DS} is estimated in accordance with Section 1613.5.4 of 2010 CBC. Based on 2010 CBC, the level of ground motion at the site is approximately 0.46 g. This value of 0.46g is in good agreement with September 2005 PSHA performed by GeoTek for the proposed residential development at the project site.

In GeoTek's study, the liquefaction susceptible layers were identified to be between existing ground surface and 58 feet bgs, based on the SPT data obtained during drilling of their borings. The results of our investigation indicate that even deeper layers, between 58 feet bgs and 75 feet bgs, may be susceptible to liquefaction, based on SPT data from the borings, and the resistance measured in CPTs (see Appendices A and C).

We evaluated the liquefaction potential and associated settlement of soils at the site using the SPT results, and equivalent SPT values from blow counts using a California Modified ring sampler, in accordance with the methodology outlined by Idriss and Boulanger (2008) using the groundwater elevation used in the GeoTek evaluation (i.e. 9 feet bgs). Included in Appendix C is the liquefaction analyses performed by us for the estimated seismically-induced settlement (assuming an M7.1 earthquake with a PGA of 0.46g).

According to the liquefaction evaluation performed by GeoTek (2005), it is anticipated that liquefied soils may experience post-liquefaction settlements of 4 to 8 inches. Our liquefaction analysis indicated that the estimated settlements due to liquefaction of the saturated Holocene age soils at the site are on the order of approximately 6.2 to 6.4-inches. In addition, based on recent in-house evaluations, estimated settlements due to liquefaction using the empirical procedures may be within 50 percent of the estimated values. As such, the estimated settlements associated with the boring should be considered within the range of 3 to 9 inches. This estimation is in agreement with the GeoTek's estimation.

We also estimated the seismically-induced settlement using the CPT data using computer program CLiq, commercially available software program (Geologismiki, 2010), which incorporates both the Ishihara and Yoshimine (1992) and Zhang et al. (2002) methodologies. Soil layers that possessed factors of safety less than 1.3 are expected to undergo volumetric strain, and as a result, settle due to liquefaction. The results of the liquefaction analysis for each CPT sounding are presented on individual plots in Appendix C.

3.2.3 Lateral Spreading

Lateral spreading involves the dislocation of the near surface soils generally along a near-surface liquefiable layer. In many cases, this phenomenon of shallow landsliding occurs on relatively flat or gently sloping ground adjacent to a "free face," such as a river embankment. Given the "weak" nature of the near surface, fine-grained sediments, shallow groundwater, liquefaction-prone soils, and the adjacent SGRC, there is a high potential for lateral spread beneath the proposed residential portion of the Site during a major earthquake in the area.

GeoTek's report addresses the potential for lateral spread. Based on the subsurface data obtained during their field investigation, they performed a preliminary estimate of liquefaction induced lateral spreading that could occur toward the SGRC. GeoTek utilized Youd, Hansen, and Bartlett's Procedure (2002) to estimate the amount lateral spreading within the Site. This procedure uses empirical equations, based on case history data. Based on their analyses, GeoTek estimated as much as 2 to 4 feet of lateral spreading could occur at a point located approximately 100 feet easterly from the SGRC as a result of a design magnitude seismic event. However, in order for this procedure to produce reliable displacement predictions (i.e., plus or minus a factor of two), the input parameters must be within the range that are set forth in the Youd (et al., 2002) research paper. In this particular case, the input parameters used by GeoTek do not fall within the range prescribed by Youd (et. al., 2002).

We also performed lateral spreading analyses using the same procedures based on the information obtained during our subsurface investigation. However, we estimated considerably more lateral spreading could potentially occur at the site. The lateral spreading that we estimated is on the order of 16 feet. We believe that the major difference between the results of the two analyses is because of the one of the input parameters, namely the fines content of subsurface native materials included in the cumulative thickness of saturated granular layers with $(N_1)_{60}$ values less than 15, used in the equations.

The subsurface data provided in GeoTek boring B-4 indicate that fines content of the materials in the saturated layers with $(N_1)_{60}$ values less than 15 is approximately 60 percent whereas, in our analysis, we used a value of 22.8 percent based on the arithmetic mean of the data obtained from our laboratory testing.

3.2.4 Seismically-Induced Landsliding

The potential for seismically-induced landsliding along the embankment/levee of the adjacent SGRC is considered moderate to high. Analytical procedures for estimating the potential lateral deformations have been developed for both sloping ground and a free face model. However, in case of a steep embankment section, such as a channel, a more applicable evaluation of lateral movement would be a seismic slope deformation analysis.

Our seismic stability of the channel embankment was evaluated using the Bray and Travasarou (2007) method for estimating earthquake-induced deviatoric slope displacements using the seismic design parameters presented in this report. In this method, pseudo-static slope stability analyses were performed to obtain the yield coefficient (k_y) of each sliding mass by applying a horizontal force that develops a Factor of Safety (FS) equal to 1.0. The initial fundamental site period (T_s) and ground motions spectral accelerations at a degraded period equal to $1.5T_s$ were then used to estimate the range of probabilistic slope displacements (mean ± 1 standard deviation).

The seismically-induced liquefaction analyses identified several layers as potentially liquefiable. A relationship between residual shear strength and corrected "clean sand" SPT blowcount $(N_1)_{60}$, as published by Seed and Harder (1990) was utilized to estimate the post liquefaction residual shear strength values for the potentially liquefiable layers. These values were used in pseudo-static slope stability analyses to obtain the yield acceleration.

Based upon the procedures outlined in Bray, J. and Travasarou, T. (2007), we determined that the potential for seismically-induced landsliding along the levee of the adjacent SGRC is considered to be moderate to high. Potential seismic slope deformation under existing conditions was estimated to range from one to four feet. Calculations for seismically-induced slope deformation are presented in Appendix D.

3.2.5 Seismically-Induced Soil Settlement

Strong ground shaking can cause settlement by allowing sediment particles to become more tightly packed, thereby reducing pore space. Unconsolidated, loosely packed alluvial deposits are especially susceptible to this phenomenon. Poorly compacted artificial fills may also experience seismically-induced settlement. Based on the subsurface data obtained from the exploratory borings drilled by GeoTek, and the two borings performed by this firm, the Holocene age alluvial soils are, for the most part, prone to seismically-induced settlement. In addition, portions of the site that are mantled with non-engineered (i.e. loose) fill soils may likely be subject to seismically-induced settlement and/ or development of ground cracking.

3.2.6 Seismically-Induced Ground Settlement of Dry Sands

Seismically-induced settlement of dry sands typically occurs with loose, relatively clean (i.e., with little or no fines) sands that are situated above the groundwater table. An analysis was performed according to criteria outlined by Tokimatsu and Seed (1987). The dynamic settlement of dry sands was evaluated using the SPT data obtained during our subsurface investigation. The same PGA and moment magnitude values used in the liquefaction analyses were used in this analysis. The results of this evaluation indicate that a maximum estimated settlement of the dry sandy soils due to the design seismic event at the site is on the order of 1 ¾ inches. Calculations for seismically-induced settlement of dry sands are presented in Appendix C.

3.2.7 Flooding/Tsunami Run-Up

Flood hazards include storm-induced flooding, and those caused by earthquakes, namely tsunami and dam failure. According to the latest (December 3, 2009) Flood Insurance Rate Map (FIRM) prepared by Federal Emergency Management Agency (FEMA), the project area does not lie within either a 100-year or 500-year flood area, or within a dam inundation area. The FIRM delineates the site as being in "Zone X," which is defined as an area of 0.2% annual chance of flood: area of 1% chance flood with average depths of less than one foot.

The greatest flooding hazard to the proposed development is that associated with tsunami inundation. A tsunami is a seismic sea-wave caused by sea-bottom deformations that are typically associated with a submarine earthquake. They are also generated by landslides, volcanic eruptions or more rarely by asteroid impact. According to the California Seismic Safety Commission (2005), the Cascadia subduction zone, which lies offshore, extending from northern California to western Canada, will produce the State's largest tsunami. The Cascadia subduction zone is similar to the Alaskan-Aleutian subduction zone that generated the M9.4, 1964 Alaska earthquake, and the Sundra subduction zone that produced the M9.3 December 2004 Sumatra earthquake.

The California Emergency Management Agency (Cal EMA), in cooperation with CGS, produced a Tsunami Inundation Map for the Seal Beach 7.5" Quadrangle (dated March 15, 2009) that depicts the project site and surrounding neighborhood lying within a tsunami inundation area. As addressed in the latest edition (FEMA 55CD, Third Edition) of the FEMA's Coastal Construction Manual (Chapter 7, Figure 7-7), a tsunami with a 90-percent probability of not being exceeded in 50 years, has the potential run-up elevation at the Site of up to 15 feet msl. With existing ground surface elevations within the proposed residential development of between 10 to 15 feet msl, the hazard from tsunamis run-up is considered significant.

3.2.8 Ground Lurching

Lurching is a phenomenon in which loose to poorly consolidated deposits move laterally as a response to strong ground shaking during an earthquake. Lurching is typically associated with soil deposits on or adjacent to steep slopes. Lurching that occurred in the Santa Monica and Santa Susana mountains during the 1994 Northridge earthquake usually was attributable to the outer two to eight feet of loose fill soils that spilled over the edges of graded pads cut into bedrock. Graded and compacted housing pads did not experience lurching during this very damaging earthquake.

Certain soils have been observed to move in a wave-like manner in response to intense seismic ground shaking, forming ridges or cracks on the ground surface. Areas underlain by thick accumulations of alluvium appear to be more susceptible to ground lurching than bedrock. Under strong seismic ground motion conditions, lurching can be expected within loose, cohesionless soils, or in clay-rich soils with high moisture content. Generally, only lightly- loaded structures such as pavement, fences, pipelines and walkways are damaged by ground lurching; more heavily loaded structures appear to resist such deformation. Ground lurching may occur where deposits of loose alluvium and/or artificial fill soils exist adjacent to the SGRC levee. In this area of the Site, ground lurching may affect lightly-loaded structures built on these materials. Therefore, the likelihood of lurching affecting the project area is considered significant.

3.2.9 Seiching

Seiching involves an enclosed body of water oscillating due to ground shaking, usually following an earthquake. Lakes and water towers are typical bodies of water affected by seiching. Given that there are no large, enclosed open bodies of water or reservoirs upgradient of the project area, the potential for seiching is nil.

3.2.10 Other Geological Hazards

Shallow Groundwater

Depth to groundwater under the Site is known to vary between about 5 to 11 feet bgs. Saturated soils and caving conditions would likely be encountered during remedial grading associated with removal and re-compaction of soils within several feet above, or at any depth below the groundwater table.

Corrosive Soils

Corrosive soils contain chemical constituents that can react with construction materials, such as concrete and ferrous metals, that may cause damage to foundations and buried pipelines. One such constituent is water-soluble sulfate which, if in high enough concentration, can react with and damage concrete. Electrical resistivity and pH level are indicators of the soil's tendency to corrode ferrous metals. According to limited laboratory testing by GeoTek, near surface soils have a relatively high pH value (8.2), and low resistivity (less than 1000 ohm-cm) indicating these soils are considered highly corrosive to ferrous metals in contact with these soils. Laboratory tests also indicate that water soluble sulfate content of 0.039 percent by weight was found within a soil sample, which is an indication of negligible sulfate exposure. As such, no particular recommendations for cement type or water ratio were deemed necessary by GeoTek to provide sulfate resistance.

Expansive Soils

Expansive soils are clay-rich soils that can undergo significant increase in volume with increased water content and significant decrease in volume with a decrease in water content. Significant changes in moisture content within moderately to highly expansive soil can produce cracking differential heave, and other adverse impacts to structures constructed on such soils. Based on the results of the laboratory test performed by GeoTek, the soils encountered in the GeoTek borings are anticipated to exhibit "low to medium" expansion potential and, therefore, the potential for expansive soils to impact new development is considered low. Placement of any clayey soils within three feet of finish grades should be avoided as stated in the GeoTek report.

Subsidence

The extraction of groundwater or oil from sedimentary source rocks can cause the permanent collapse of pore space that was previously occupied by the removed fluid. The compaction of subsurface sediments resulting from fluid withdrawal can and has caused the ground surface overlying fluid reservoirs to subside. If sufficiently great, the subsidence can cause significant damage to nearby engineered structures. As stated earlier in this report, the Site is not situated within an active or historic oil or gas field. The nearest major oil producing areas in the vicinity of the Site are the Seal Beach and Wilmington fields located about one mile to the north and south, respectively.

However, during the major oil and gas production years between 1928 and 1970, oil withdrawals within the Wilmington field produced as much as 30 feet of land subsidence within a bowl-shaped area centering on downtown Long Beach. As a consequence, the Site experienced less than about 1.5 feet of land subsidence during this time. However, beginning in the late 1950s, water injection into various wells was employed to arrest the subsidence, which produced about 0.1 feet of rebound (recovered elevation) at the Site, as measured during the period 1966-1970. There is no indication that the Site has experienced any significant subsidence or rebound since 1970, and significant changes in elevation at the Site are not anticipated to pose a significant hazard to the project, barring such extractions in the future.

There has been no measurable land subsidence documented within the nearby Seal Beach/ Hellman Ranch oil field during the last 60 years.

Soil Erosion

Soil erosion is most prevalent in unconsolidated alluvium and surficial soils, which are prone to downcutting, sheetflow, and slumping and bank failure during and after heavy rainstorms. Strong wind forces can also produce varying amounts of soil erosion of unconsolidated surficial soils. However, long-term shoreline erosion can be caused by a number of factors, such as rising sea levels, reduced sediment supply to the coast, dredging of the nearby SGRC and increased incidence of intensity of storms.

The short-term effects of soil erosion during rough grading for the project are not considered significant, given that the project site is essentially flat, and does not possess site conditions necessarily conducive to soil erosion. It is anticipated that temporary screen walls will be placed around the perimeter of the project during rough grading, which should help mitigate wind-related soil erosion. Therefore, the potential for short-term soil erosion is considered nil.

Given the inherent uncertainties regarding long-term shoreline erosion, impacts on soil erosion within the project area cannot be fully assessed.

Sloughing or Caving of Excavations

During construction for the project excavations associated with remedial grading/ ground stabilization and underground utilities will encounter unconsolidated/noncohesive artificial fill, as well as saturated paralic soils. If unsupported, these soils will be subject to sloughing and caving, hence creating a short-term hazard to construction workers and equipment.

Prime Farmland

Based on a review of historic aerial photographs dating back to 1927, there is no indication that the Site was used for farming or other agricultural purposes.

4.0 THRESHOLDS OF SIGNIFICANCE

Earth resource and/or topographic impacts resulting from the proposed project could be considered significant if any of the following occur:

- Exposure of people or property to substantial geological hazards, such as flooding due to dam or reservoir failure, landslides, mudslides, ground failure or similar hazards; or soil and/or seismic conditions so unfavorable that they could not be overcome by design using reasonable construction and/or maintenance practices;

- Location of a structure within a mapped hazard area or within a structural setback zone;

- Location of a structure within an Alquist-Priolo Fault-Rupture Hazard Zone, or within a known active fault zone, or an area characterized by surface rupture that might be related to a fault;

- Triggering or acceleration of geologic processes, such as landslides or erosion that could result in slope or embankment/levee failures;

- Substantial irreversible disturbance of the soil materials at the site or adjacent sites, such that their use is compromised;

- Modification of the surface soils such that abnormal amounts of windborne or waterborne soils are removed from the Site;

- Earthquake-induced ground shaking capable of causing ground rupture, liquifaction, soil settlement, landsliding resulting in substantial damage to people and/or property;

- Deformation of foundations by expansive soils (those characterized by shrink/swell potential) or collapsible soils; and

- Modification of the on-site topography (i.e. grading) in a manner that results in decreased stability for adjacent residential enclaves.

5.0 IMPACTS

The level of geotechnical and landform information contained herein is adequate to analyze the potential project effects on earth resources and landforms, and to determine appropriate mitigation measures for the proposed development. In accordance with CEQA case law, these later additional refinements are not a deferral of mitigation. Rather, it is a design refinement, consistent with the commitment to mitigation included in this EIR.

Essentially, there are a number of impacts to the current physical/geological setting that can generally be expected from grading and development activities associated with the proposed development.

5.1 EFFECTS FOUND NOT TO BE SIGNIFICANT

Based on the results of the information reviewed for this study, those geologic hazards that are not considered to represent significant impacts due to their absence or low potential within the Site include ground surface rupture associated with active faulting, short-term soil erosion, expansive soils, subsidence, topography, and loss of prime farmland.

5.2 POTENTIALLY SIGNIFICANT CONSTRUCTION-RELATED IMPACTS

The most significant potential impacts to the project are those resulting from strong seismically-induced ground motion, earthquake-induced soil settlement associated with liquefaction of saturated, loose paralic soils, as well as loose fill soils, lateral spreading, seismically-induced landsliding, tsunami run-up, and ground lurching. Other geologic-related impacts include shallow saline groundwater, sloughing/caving of excavations, and soils that are considered to be severely corrosive to metallic pipes.

5.2.1 Strong Seismically-Induced Ground Motion

Given the proximity to major active faults, severe ground motion should be expected at the site. All structures associated with the proposed development should be designed to withstand the "design-level" earthquake as set forth in the latest edition of the CBC. Potential adverse impacts to new structures due to strong, seismically-induced, vibratory ground motion can be reduced to a less-than-significant level with proper seismic design.

5.2.2 Liquefaction and Settlement Prone Soils

Saturated paralic soils are subject to varying amounts of liquefaction-induced settlement resulting from strong seismically-induced ground motions. The impact to structures having footings or structural elements founded in these soils could be significant unless mitigated. Typical mitigation concepts would include the following:

- Over-excavation and re-compaction of the liquefaction-prone soils;
- In situ soil densification such as vibro-flotation, vibro-replacement (i.e. stone columns);
- Injection grouting; or
- Deep soil mixing.

In addition, unsaturated, unconsolidated paralic and uncompacted artificial fill soils are also prone to seismically-induced settlement. It is anticipated that the future geotechnical engineering studies to be performed for the proposed development will further evaluate the nature and extent of these types of soils.

5.2.3 Seismically-Induced Lateral Spreading

The presence of underlying liquefaction-prone soils and the site location relative to the SGRC poses a significant risk of seismically-induced lateral spread. Significant distress to both above- and below-ground structures would occur in the event of this form of seismically-induced landsliding. Our analyses suggest that lateral spread could impact approximately 80 percent, or more, of the proposed residential area, as well as the southern one-quarter and western central portion of the Site. The actual extent of lateral spread should be performed as part of the site-specific geotechnical investigation for the project. The reinforcement of soils within the potential zones of lateral spread will be necessary. The methods to mitigate lateral spread are similar to those presented above to mitigate soils prone to liquefaction.

5.2.4 Seismically Induced Landsliding

Given the perceived, weak nature of the soils underlying the levee, our analyses indicate that the levee, and portions of the Site along the eastern side of the levee, is subject to landsliding during a moderate to strong seismic event in the area. The reinforcement of soils within the potential zones of landsliding will be necessary. As with the liquefaction and lateral spreading issues, this hazard should be further evaluated as part of the site-specific geotechnical investigation for the project. The methods to mitigate seismically-induced landsliding are similar to those presented above to mitigate liquefaction and lateral spreading.

5.2.5 Seismically-Induced Soil/Ground Settlement

The susceptibility of Holocene-age paralic soils, as well as non-engineered (i.e. loose) fill soils to seismically-induced settlement presents a significant impact to Site. The methods to mitigate seismically-induced soil settlement are similar to those presented above to mitigate liquefaction and lateral spreading.

5.2.6 Ground Lurching

Ground lurching may occur where deposits of loose alluvium and/or artificial fill soils exist adjacent to the SGRC levee. The methods to mitigate ground lurching are similar to those presented above to mitigate soils prone to liquefaction, lateral spread and seismically-induced landsliding.

5.2.7 Tsunami Run-Up

Present building codes and guidelines do not adequately address the impacts of tsunami on structures. FEMA's latest Coastal Construction Manual (55CD) provides design and construction guidance for structures built in coastal areas, which address seismic loads for coastal structures and provides information on tsunami and associated loads for tsunami run-up. However, as pointed out by the Certified Structured Settlement Consultant (CSSC) (2005), the authors of the Coastal Construction Manual concluded that tsunami loads are too great and that, in general, it is not feasible or practical to design "normal" structures to withstand these loads.

5.2.8 Shallow Groundwater

Depending upon the construction methods employed, dewatering may be required in order to safely excavate the Site just above and below groundwater, which will likely require some form of lateral support. The saline groundwater pumped from the dewatering wells will need to meet National Pollutant Discharge Elimination System (NPDES) permit requirements before it is discharged.

5.2.9 Corrosive Soils

Near surface soils are highly corrosive to metals in contact with these soils. It is anticipated that the future geotechnical engineering studies to be performed for the proposed development will further evaluate the nature and extent of these types of soils. At a minimum, buried metal piping should be protected with suitable coatings, wrapping, or seals; and a corrosion engineer should be consulted during future, site-specific geotechnical studies.

5.2.10 Topography

Grading activities associated with the development and construction of new buildings, underground utilities, and associated parking areas would create very little change to the current topography. The greatest changes to existing topography would occur from construction of residential structures.

Only by avoidance can impacts to topography related to the taller building(s) be mitigated and/or reduced to a less-than-significant level.

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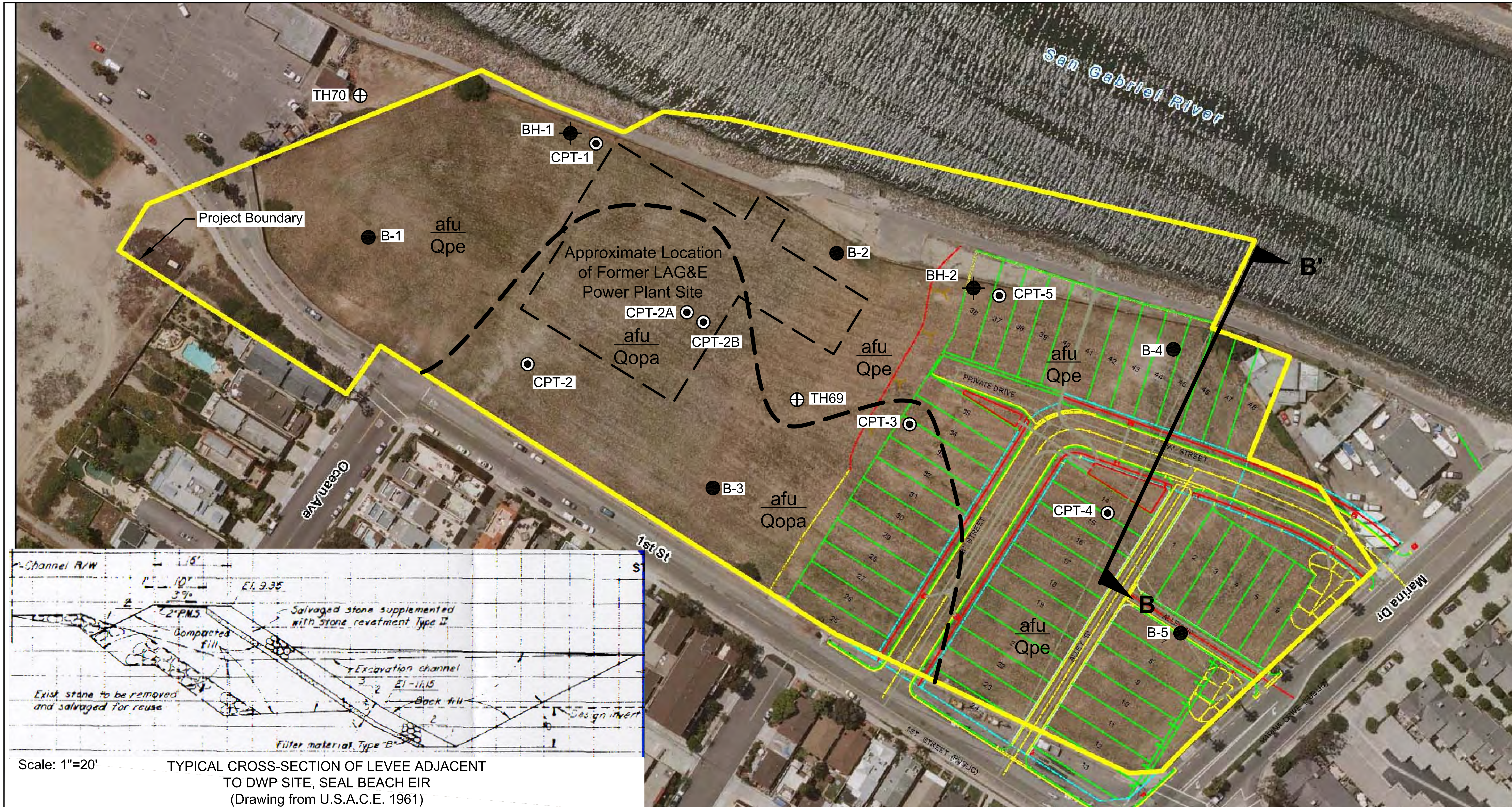
AERIAL PHOTOGRAPHS REVIEWED

Source	Date of Photographs	Flight No/ Frame No.	Scale
U.C. Santa Barbara	1927	C-300/ M210, M225 and M226	1:18,000
U.C. Santa Barbara	5/7/40	WATSON-261/ 0 and 1	1:4,800
U.C. Santa Barbara	3/21/52	C-300 PAI 33/ 2, 3, and 4	1:480
U.C. Santa Barbara	6/20/63	PAI214V-A/ 91	1:6,000
MAPTECH/ USGS	1/4/04	3318-F1-OF-PDQ	1:3,600



Scale: 1"=20'

TYPICAL CROSS-SECTION OF LEVEE ADJACENT
TO DWP SITE, SEAL BEACH EIR
(Drawing from U.S.A.C.E. 1961)

Explanation

- Cone penetration test (this investigation), approximately located
- Exploratory rotary wash boring (this investigation), approximately located
- Exploratory hollow-stem auger boring by GeoTek, Inc. (2005), approximately located
- Exploratory bucket-auger boring by U. S. Army Corps of Engineers (U.S.A.C.E. 1961), approximately located

B B' Slope stability cross section

Geologic contact, approximately located

afu/Qpe Undocumented artificial fill overlying Holocene-age paralic deposits

afu/Qopa Undocumented artificial fill overlying late to middle Pleistocene-age paralic deposits

DRAFT

0 50 100
Approximate Scale in Feet

Basemap modified from aerial photograph dated 2010 provided by RBF Consulting

SITE PLAN AND GEOLOGIC MAP
Seal Beach DWP Site EIR Study
Seal Beach, California

By: pah	Date: 06/27/11	Project No.: NB11161340
D. Scott Magorien, C.E.G. Consulting Engineering Geologist		Figure 1

Plot Date: 06/24/11 - 6:11pm, Plotted by: pat.herring
Drawing Path: Y:\NB11161340\acad\, Drawing Name: tb_geology and sample points.dwg

APPENDIX A

Exploratory Boring Logs and CPTs

MAJOR DIVISIONS		LTR	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixture, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SAND	SW	Well-graded sands or sand with gravel, little or no fines
		SP	Poorly-graded sands or sand with gravel, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures

MAJOR DIVISIONS		LTR	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS LL<50	ML	Inorganic silts and very fine sand, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	SILTS AND CLAYS LL>50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of hogh plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils

SAMPLE COLUMN SYMBOLS

Standard penetration test (SPT)

California Split Spoon Sample

Modified California Split Spoon

Sample Interval

Piston Sample

Continuous soil or rock core

NR No recovery

BLOWS/FOOT - Summation of blow counts for deepest 12 inches is sampling interval
RQD% - Rock quality destination in percent

DESCRIPTION COLUMN SYMBOLS

Dashed lines separating soil strata represent inferred boundaries between sampled intervals or no recovery intervals and may be distinct or gradual transitions

Solid lines represent distinct or gradual boundaries observed within sampled intervals

Description right of bracket symbol represents soil conditions within the depth interval defined by the bracket length

Description right of arrow symbol represents soil conditions to the next deeper boundary line unless otherwise noted

Water level at time of drilling

Water level after at least 12 hours from time of drilling

LABORATORY TEST ABBREVIATIONS

ATT	Atterberg Limits	EI	Expansion Index	SE	Sand Equivalent
COLL	Collapse Potential	SIEVE	Grain Size Analysis	SG	Specific Gravity
COMP	Compaction	LL	Liquid Limit	TX	Triaxial Test
CON	Consolidation	PERM	Permeability	UC	Unconfined Compression Test
CORR	Corrosion	PI	Plastic Index	#200	No. 200 Wash Sieve Analysis
DS	Direct Shear	R-VALUE	R-Value		

NOTES

1. Soil descriptions are in accordance with the USCS as set forth by ASTM D2488-90 "Standard Practice for Description and Identification Soil (Visual-Manual Procedure)."
2. Soil color described according to Munsell Soil Color Chart. Rock color described according to Munsell Rock-Color Chart
3. Soil descriptions in these borings are generalized representations and based upon visual classification of cuttings and/or samples during drilling. Descriptions and related information in these borings depict subsurface conditions at the specific location and at the time of drilling only. Soil conditions at other locations may differ from conditions observed at the boring locations. Also, soil and groundwater conditions may change with time at these locations.

D. Scott Magorien, C.E.G. 1290 Engineering Geologist	**EXPLANATION OF BORING LOGS** SEAL BEACH DWP SITE EIR Seal Beach, California	Project No. NB11161340

PROJECT: SEAL BEACH DWP SITE EIR	**Log of Boring No. BH1**

BORING LOCATION: Refer to Figure 1 - Site Geologic Map

DATE STARTED: 5/4/11	DATE FINISHED: 5/5/11	NOTES:
DRILLING METHOD: mud rotary		Drilling Contractor: Gregg Drilling & Testing, Inc.
HAMMER WEIGHT: 140 lb	DROP: 30 in.	Drilling Equipment: Mobile B53
SAMPLER: SPT & modified California		Logged By: V Robino

ELEV. (feet)	DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: not surveyed; datum is ground surface	LABORATORY TESTS Moisture Content (%)	Dry Density (pcf)	Other Tests
	0–2.5	BH1-1	NR	2 3 4	SANDY SILT (ML): brown (10YR 4/3), moist, ~60% fines, ~40% fine sand, trace medium and coarse sand, low plasticity, rapid dilatancy, low toughness, soft, medium dry strength, roots upper 3", fine gravel 0.5-0.75'			
	2.5–4	BH1-2		5 9 12	CLAYEY SAND (SC): olive brown (2.5Y 4/3), moist, ~70% fine to medium sand, ~30% medium plasticity fines shell fragments			
	4–5.25				POORLY GRADED SAND (SP): olive brown (2.5Y 4/3), wet, ~95% fine to medium sand, ~5% fines, shell fragments			
	5.25–8.25	BH1-3	NR	14 17 14	SILTY SAND (SM): dark grayish brown (2.5Y 4/2), ~85% fine to medium sand, ~15% low plasticity fines, abundant shell fragments @6' copper fragments observed in top of SPT sample, no loss of drilling fluid			
	8.25–11.25	BH1-4	NR	7 10 12	POORLY GRADED SAND (SP): dark grayish brown (2.5Y 4/2), ~95% fine sand, ~5% fines, shell fragments LEAN CLAY (CL): black (2.5Y 2.5/1) LEAN CLAY (CL): brown (10YR 4/3)			
	11.25–12.5				POORLY GRADED SAND with SILT (SP-SM): olive gray (5Y 4/2), ~90% fine sand, ~10% low plasticity fines, shell fragments			
	12.5–14.25	BH1-5	NR	5 7 4	POORLY GRADED SAND (SP): very dark greenish gray (10G 3/1), ~95% fine sand, ~5% fines, abundant shell fragments and plant matter			
	14.25–15				SILTY SAND (SM): olive brown (2.5Y 4/3), ~80% fine sand, ~20% low plasticity fines			

PROJECT: SEAL BEACH DWP SITE EIR

Log of Boring No. BH1 (cont'd)

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other Tests
				6	SILTY SAND (SM): continued			
		BH1-6		8				sieve
	16		NR	9				
	17							
	18			3				
		BH1-7		2				
	19		NR	5	CLAYEY SAND (SC): olive brown (2.5Y 4/4), ~65% fine sand, ~35% medium plasticity fines			
	20				POORLY GRADED SAND (SP): olive brown (2.5Y 4/4), ~95% fine to medium sand, ~5% fines			
	21			10				
		BH1-8		11				
	22			13				
	23							
	24			7				
		BH1-9		12				
	25		NR	12				
	26							
	27							
	28				POORLY GRADED SAND with SILT (SP-SM): olive brown (2.5Y 4/4), ~90% fine to medium sand, ~10% fines			
	29			20				
		BH1-10		24		23.4	103.1	
	30			28	trace coarse sand			
				4	LEAN CLAY (CL): light olive brown (2.5Y 5/6), ~100% fines			
	31	BH1-11		7	LEAN CLAY (CL): light olive brown (2.5Y 5/6)			
				12	LEAN CLAY (CL): light olive brown (2.5Y 5/6)			
	32							

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PROJECT: SEAL BEACH DWP SITE EIR

Log of Boring No. BH1 (cont'd)

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/ Foot	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other Tests
					POORLY GRADED SAND with SILT (SP-SM): continued			
					POORLY GRADED SAND (SP): ~95% fine to medium sand, ~5% fines			
	33			9				
		BH1-12		13				
	34			16				
	35				olive brown (2.5Y 4/4)			
	36			11				
		BH1-13		14				
	37		NR	19				
	38				fine to medium sand			
	39			13				
	40	BH1-14		15				
				17				
	41							
	42			10				
		BH1-15		13				
	43		NR	14				
	44				POORLY GRADED SAND with SILT (SP-SM): olive brown (2.5Y 4/4), ~90% fine to medium sand, ~10% low plasticity fines			
	45			12				
		BH1-16		14				#200 = 8.9%
	46		NR	17				
	47							
	48							
	49							

PROJECT: SEAL BEACH DWP SITE EIR

Log of Boring No. BH1 (cont'd)

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other Tests
					POORLY GRADED SAND with SILT (SP-SM): continued			
	50			7				
		BH1-17		5	LEAN CLAY (CL): olive brown (2.5Y 4/4), ~100% fines, medium plasticity, no dilatancy, high dry strength, firm			
	51			6	CLAYEY SAND (SC): olive brown (2.5Y 4/4), ~70% fine to coarse sand, predominantly fine to medium, ~30% medium plasticity fines 51.25-51.5' SANDY LEAN CLAY (CL)			
	52							
	53				POORLY GRADED SAND (SP): dark grayish brown (2.5Y 4/2), ~95% fine to medium sand, ~5% fines			
	54							
	55			17				
		BH1-18		25				
	56		NR	21				
	57							
	58							
	59							
	60			24				
		BH1-19		29				
	61			33				
	62							
	63							
	64							
	65			22	olive gray (5Y 4/2)			
		BH1-20		24	layering/iron oxide staining			
	66							

PROJECT: SEAL BEACH DWP SITE EIR	Log of Boring No. BH1 (cont'd)

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other Tests
				30	POORLY GRADED SAND (SP): continued			
	67							
	68							
	69							
	70			22				
		BH1-21		25				
	71		NR	34				
	72							
	73				olive gray (5Y 4/2), layering absent			
	74							
	75			28				
		BH1-22		39				
	76		NR	50/5"				
	77				Bottom of boring at 76.5' bgs Groundwater encountered at a depth of approximately 11 feet below ground surface in adjacent CPT-1. Boring backfilled with grout.			
	78							
	79							
	80							
	81							
	82							
	83							

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PROJECT: SEAL BEACH DWP SITE EIR

Log of Boring No. BH2

BORING LOCATION: Refer to Figure 1 - Site Geologic Map

DATE STARTED: 5/5/11	DATE FINISHED: 5/5/11	NOTES: Drilling Contractor: Gregg Drilling & Testing, Inc. Drilling Equipment: Mobile B53 Logged By: V Robino
DRILLING METHOD: mud rotary		
HAMMER WEIGHT: 140 lb	DROP: 30 in.	
SAMPLER: SPT & modified California		

Surface Elevation: not surveyed; datum is ground surface

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other Tests
	0–2				SILTY SAND (SM): light olive brown (2.5Y 5/6), moist, ~65% fine to medium sand, ~35% low to medium plasticity fines, trace fine gravel, roots			
	2–7.3				CLAYEY SAND (SC): brown (7.5YR 5/4), ~65% fine to medium sand, ~35% medium plasticity fines			
	4–5.5	BH2-1	NR	2 3 2	granitic cobble			
	7.3–12				LEAN CLAY (CL): dark greenish gray (10G 4/1), ~95% fines, ~5% fine sand, low to medium plasticity, no dilatancy, low toughness, very soft, high dry strength			
	9–10.5		NR	0 0 2				
	12–14.5	BH2-2		1 1 1	LEAN CLAY with SAND (CL): dark greenish gray (10G 4/1), ~85% fines, ~15% fine sand, low to medium plasticity, no dilatancy, low toughness, soft, high dry strength	30.8	92.5	LL = 38 PI = 19
	14.5–15				SANDY SILT (ML): see next page			

ELEV. (feet)	DEPTH (feet)	Sample No.	Blows/Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other Tests
	16	BH2-3	1, 1, 3	SANDY SILT (ML): very dark greenish gray (10Y 3/1), ~55% fines, ~45% fine sand, trace organic matter, low plasticity, rapid dilatancy, low toughness, soft, low dry strength 16-16.25' LEAN CLAY (CL): very dark greenish gray (10Y 3/1), ~100% fines, medium plasticity, no dilatancy, low toughness, high dry strength, very soft			
	17–18	BH2-4	2, 5, 5	CLAYEY SAND (SC): very dark gray (N 3/), ~60% fine to medium sand, ~40% medium plasticity fines, trace organic matter very dark greenish gray (10Y 3/1), ~90% sand, ~10% fines			
	19			SILTY SAND (SM): very dark greenish gray (10Y 3/1), ~55% fine sand, ~45% low plasticity fines, organic matter			
	20–21			CLAYEY SAND (SC): very dark gray (N 3/), ~70% fine to medium sand, ~30% medium plasticity fines, organic matter			
	22	BH2-5	0, 0, 0	LEAN CLAY with SAND (CL) ~80% sand, ~20% fines			sieve
	23–25	BH2-6	8, 12, 13	SILTY SAND (SM): very dark greenish gray (10Y 3/1), ~60% fine to medium sand, ~40% low plasticity fines			
	26–28	BH2-7	3, 3, 5 (NR)	dark greenish gray (10Y 4/1), dark greenish gray (10Y 4/1), ~80% fine sand, ~20% fines, trace medium sand			sieve
	29–32	BH2-8	6, 10, 12 (NR)	POORLY GRADED SAND (SP): very dark greenish gray (10Y 3/1), ~95% fine to medium sand, ~5% fines			

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Log of Boring No. BH2 (cont'd)

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other Tests
					POORLY GRADED SAND (SP): continued			
	33	BH2-9		10, 7, 8	dark olive brown (2.5Y 3/3)			
	34				FAT CLAY (CH): olive gray (5Y 5/2), ~100% fines, high plasticity, no dilatancy, low toughness, soft to firm, high dry strength abundant shells and fragments olive (5Y 4/4), ~95% fines, ~5% fine sand			
	35–36	BH2-10		10, 12, 15		33.4	88.7	LL = 62 PI = 37
	38				SILTY SAND (SM): olive (5Y 4/3), ~60% fine sand, ~40% nonplastic to low plasticity fines			
	39–40	BH2-11		10, 8, 13				
	41				POORLY GRADED SAND (SP): olive brown (2.5Y 4/3), ~95% fine to medium sand, ~5% fines			
	42–43	BH2-12		17, 20, 20				
	44				SILTY SAND (SM): dark grayish brown (2.5Y 4/2), ~80% fine to medium sand, ~20% low plasticity fines			
	45–46	BH2-13	NR	6, 8, 13				
	47–49							

PROJECT: SEAL BEACH DWP SITE EIR

Log of Boring No. BH2 (cont'd)

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other Tests
					POORLY GRADED SAND (SP): olive gray (5Y 4/2), ~95% fine to medium sand, ~5% fines			
	50	BH2-14		17, 19, 20				
	51							
	52							
	53							
	54							
	55	BH2-15	NR	16, 21, 29	abundant shell fragments			
	56				SILTY SAND (SM): olive gray (5Y 4/2), ~65% fine sand, ~35% low plasticity fines			
	57							
	58				POORLY GRADED SAND (SP): olive (5Y 4/3), ~95% fine to medium sand, ~5% fines			
	59							
	60	BH2-16		20, 26, 43				
	61							
	62							
	63				LEAN CLAY with SAND (CL): olive (5Y 4/3), ~85% fines, ~15% fine sand, medium plasticity, rapid dilatancy, low toughness, low dry strength, firm			
	64							
	65	BH2-17		10, 18				#200 = 84.3%
	66							

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PROJECT: SEAL BEACH DWP SITE EIR

Log of Boring No. BH2 (cont'd)

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other Tests
				30	LEAN CLAY with SAND (CL): continued			
	68.5				POORLY GRADED SAND (SP): olive (5Y 4/3), ~95% fine to medium sand, ~5% fines			
	70–71.5	BH2-18		14 20 20				
	71.5				SILTY SAND (SM): olive (5Y 4/3), ~60% fine sand, ~40% low plasticity fines			
	73.5				~85% sand, ~15% fines			
	75–76.5	BH2-19	NR	7 12 20	SANDY SILT (ML): ~70% fines, ~30% fine sand, low plasticity			#200 = 66.8%
	75.8				LEAN CLAY (CL) iron oxide layering			
	76.5				Bottom of boring at 76.5' bgs Groundwater encountered at a depth of approximately 11 feet below ground surface in adjacent CPT-5. Boring backfilled with grout.			

Depth scale: 67, 68, 69, 70, 71, 72, 73, 74, 75, 76, 77, 78, 79, 80, 81, 82, 83

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SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

Seal Beach DWP Site EIR
Ocean Avenue & 1st Street
Seal Beach, CA
May 4, 2011

Prepared for:

Mr. Scott Magorien
D. Scott Magorien Consulting
510 Superior Avenue, Ste 200
Newport Beach, CA 92663-3627
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Prepared by:

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Huntington Beach, CA 92649-1518
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TABLE OF CONTENTS

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the Seal Beach DWP Site EIR project located at Ocean Avenue & 1st Street in Seal Beach, California. The work was performed by Kehoe Testing & Engineering (KTE) on May 4, 2011. The scope of work was performed as directed by D. Scott Magorien Consulting personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at seven locations to determine the soil lithology. Groundwater measurements and hole collapse depths provided in **TABLE 2.1** are for information only. The readings indicate the apparent depth to which the hole is open and the apparent water level (if encountered) in the CPT probe hole at the time of measurement upon completion of the CPT. KTE does not warranty the accuracy of the measurements and the reported water levels may not represent the true or stabilized groundwater levels.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	75	Hole open to 10.0 ft (dry)
CPT-2	75	Groundwater @ 10.0 ft
CPT-2A	8	Refusal, hole open to 7.0 ft (dry)
CPT-2B	6	Refusal, hole open to 5.0 ft (dry)
CPT-3	75	Hole open to 8.0 ft (dry)
CPT-4	75	Groundwater @ 11.0 ft
CPT-5	75	Groundwater @ 5.5 ft

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed
- Pore Pressure Dissipation (at selected depths)

The above parameters were recorded and viewed in real time using a portable computer and stored on a diskette for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the CPT Classification Chart (Robertson, 1986) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

Output from the interpretation program CPTINT provides averaged CPT data over one-foot intervals. The CPTINT output includes Soil Classification Zones, SPT N Values and Undrained Shear Strength (Su). A summary of the equations used for the tabulated parameters is provided in the CPTINT Correlation Table in the Appendix.

The interpretation of soils encountered on this project was carried out using correlations developed by Robertson et al, 1986. It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgment and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

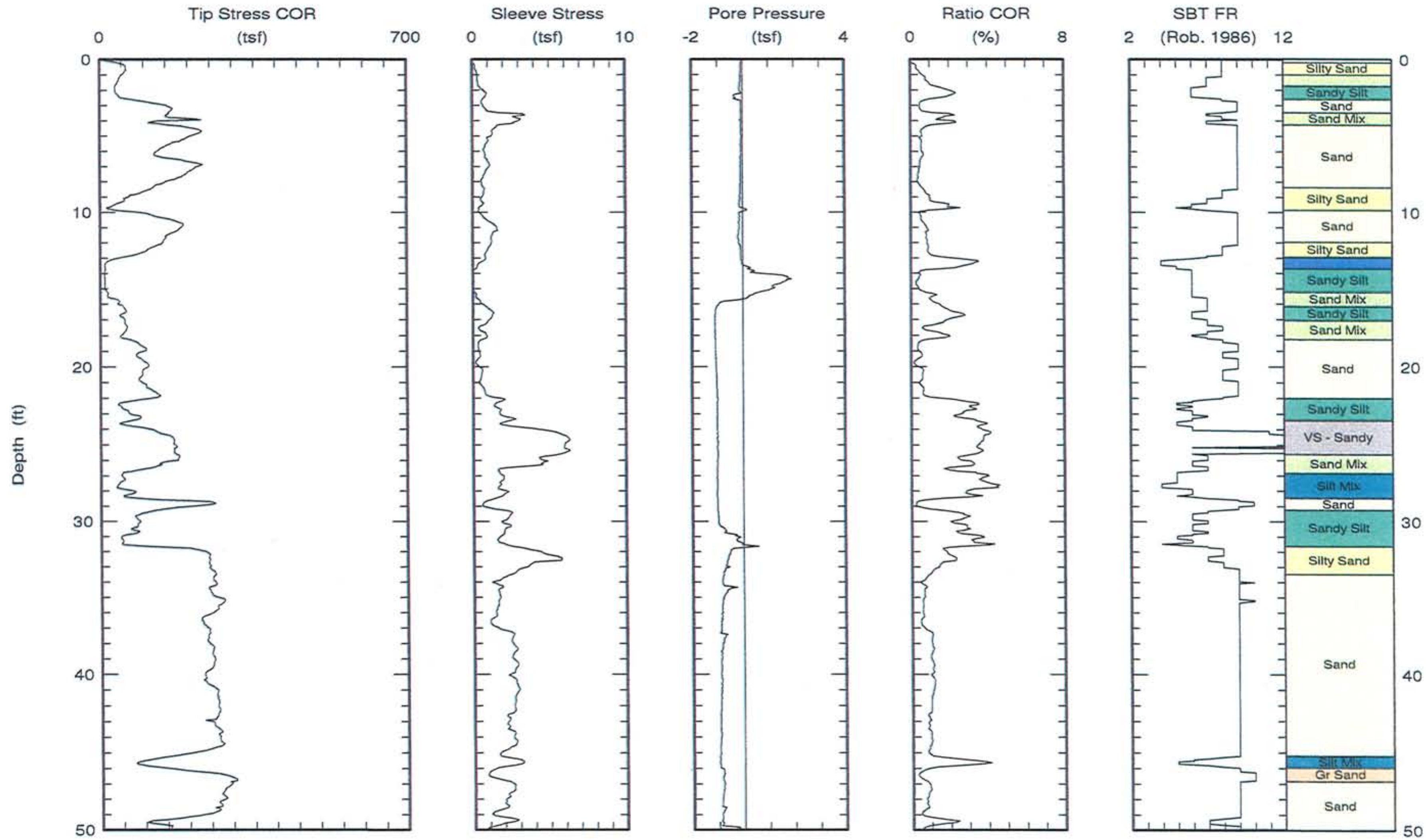
KEHOE TESTING & ENGINEERING

Richard W. Koester, Jr.
General Manager

05/19/11-dp-1-1526

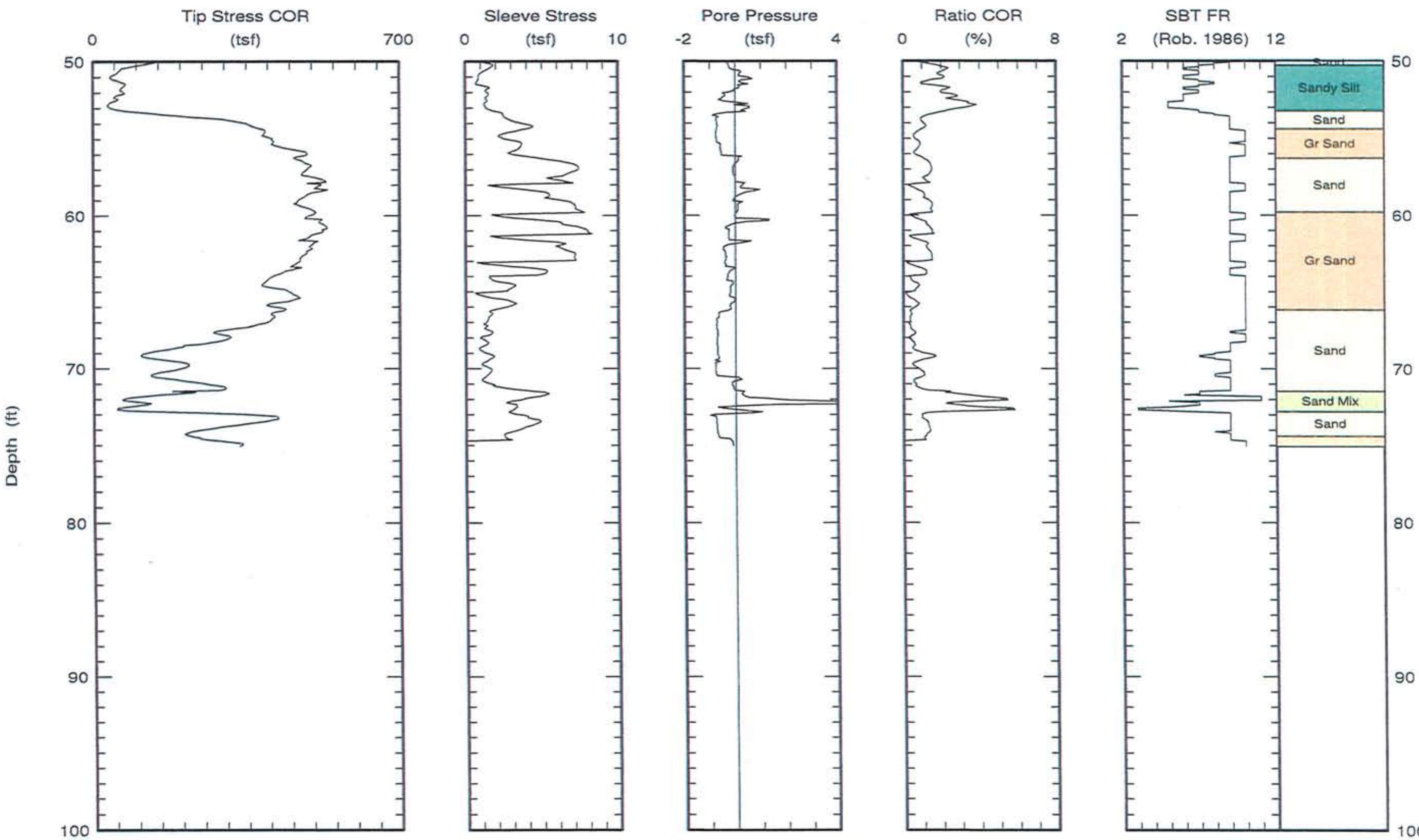
APPENDIX

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-1 Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



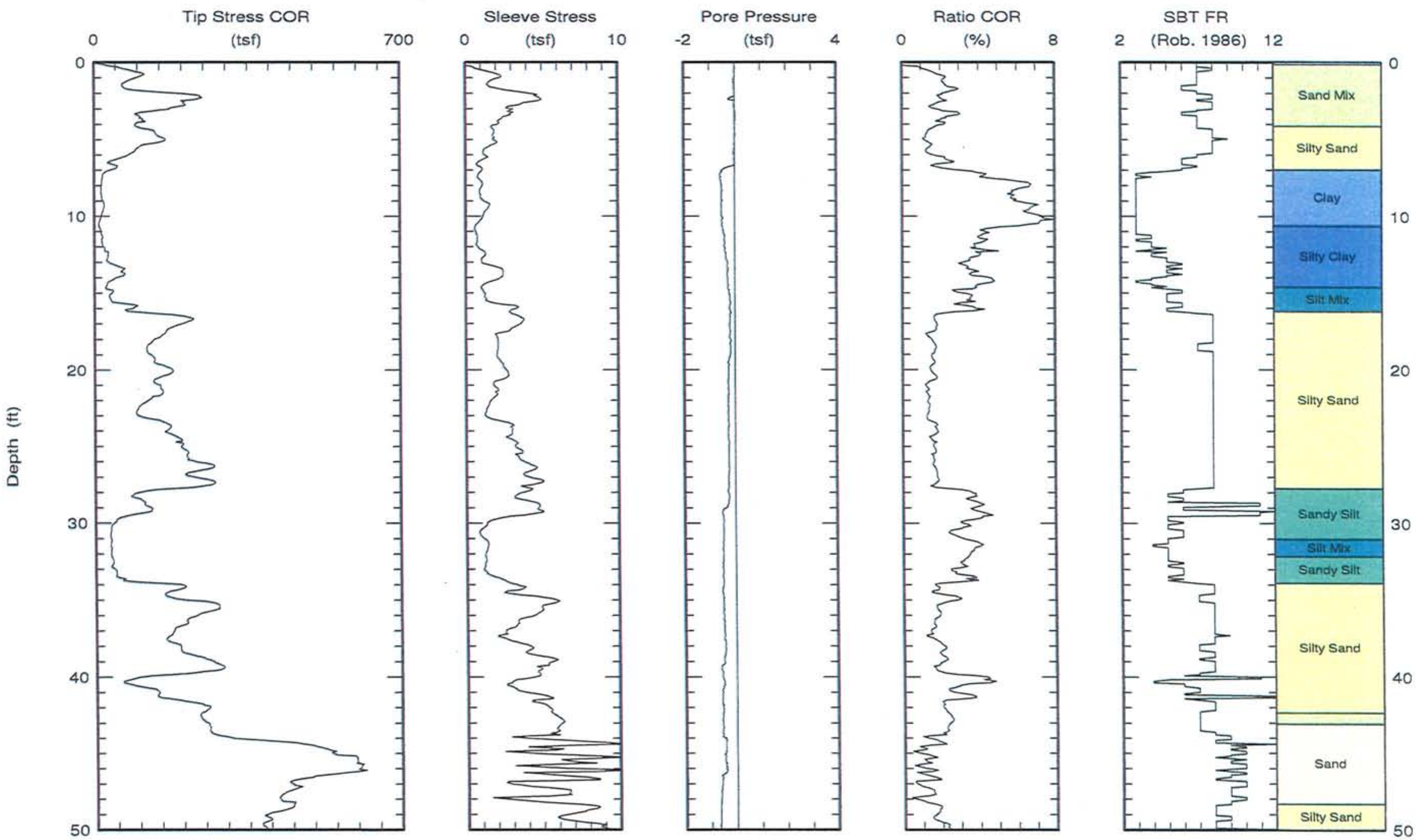
Maximum depth: 75.07 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-1 Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



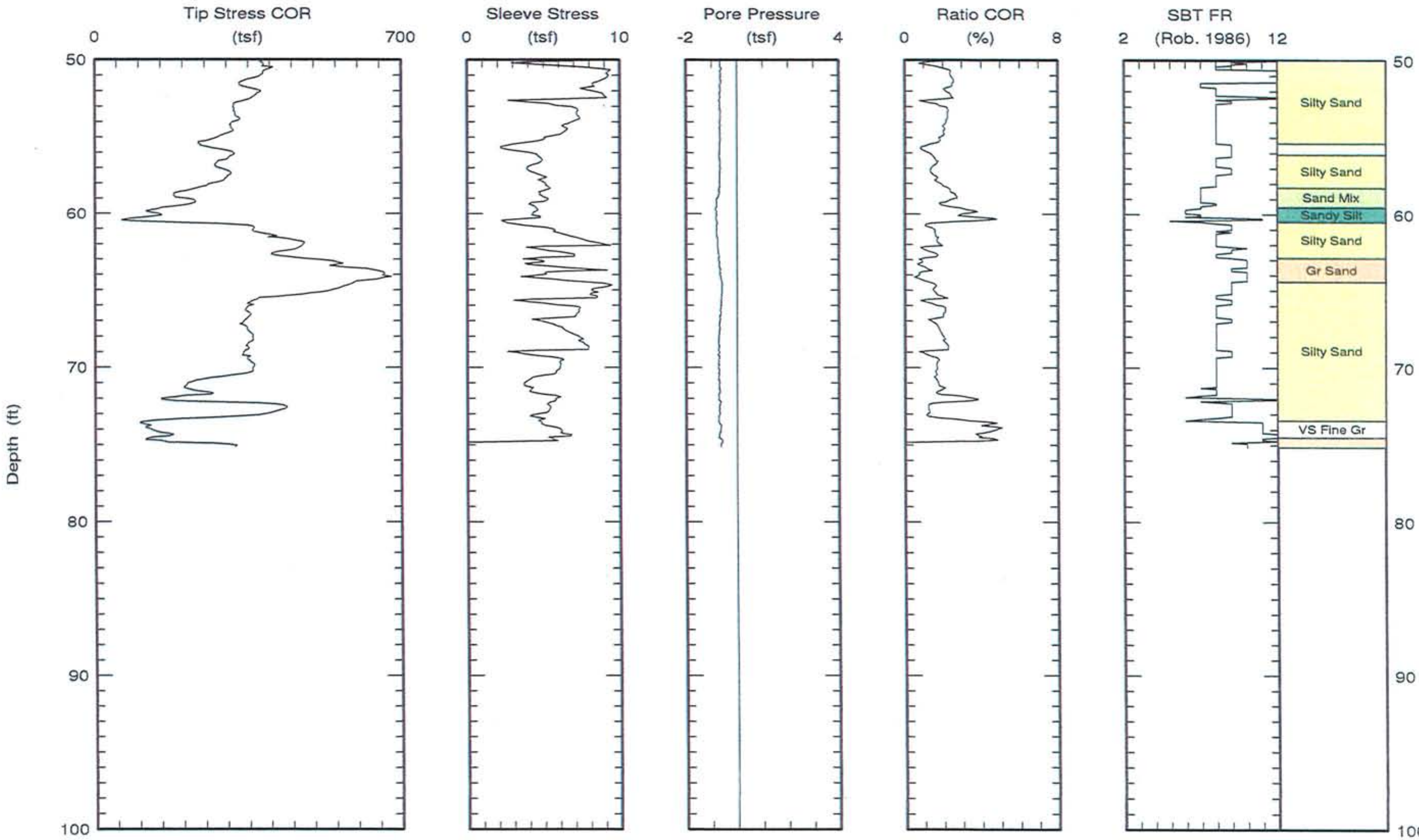
Maximum depth: 75.07 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-2 Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



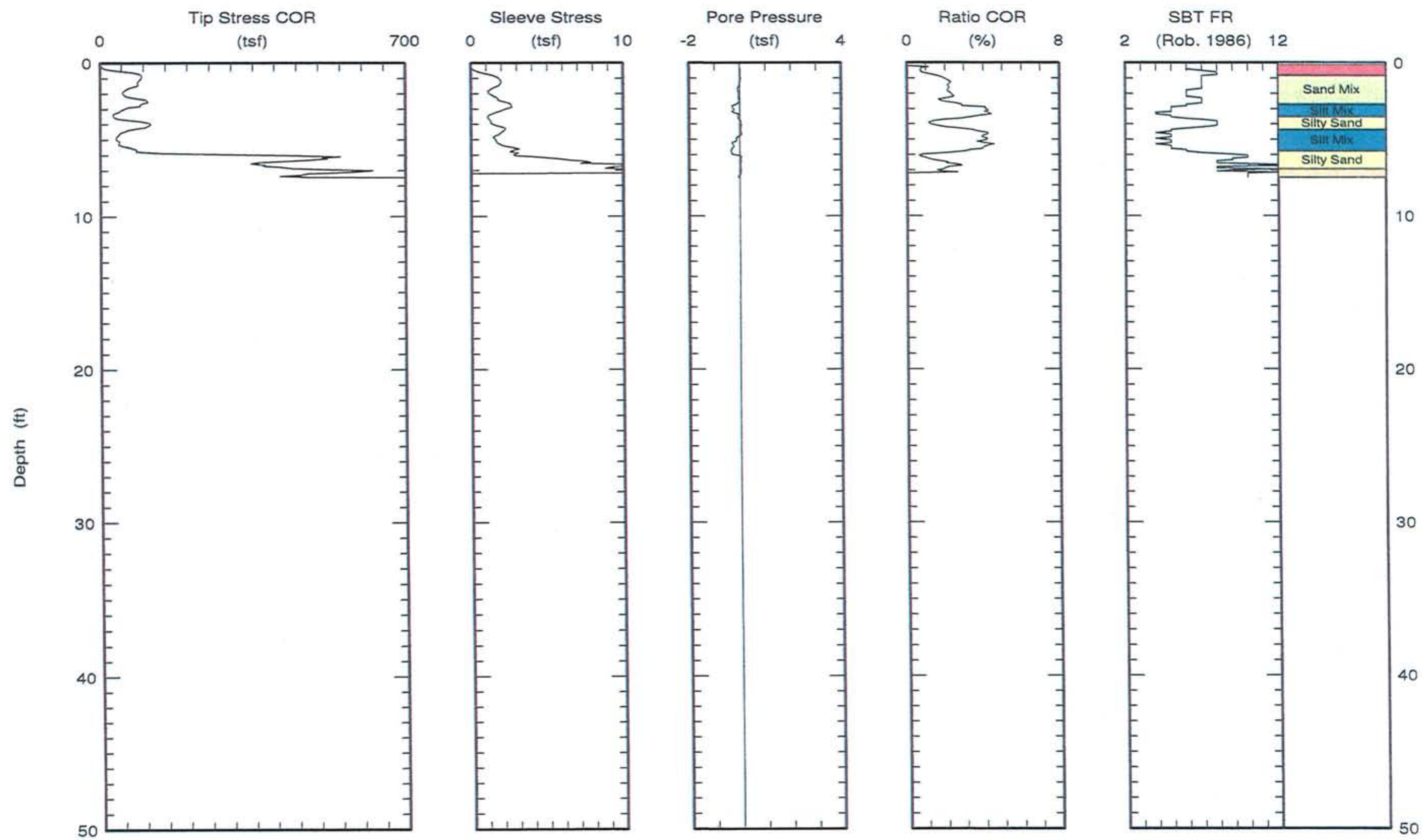
Maximum depth: 75.14 (ft)
Page 1 of 2

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-2 Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



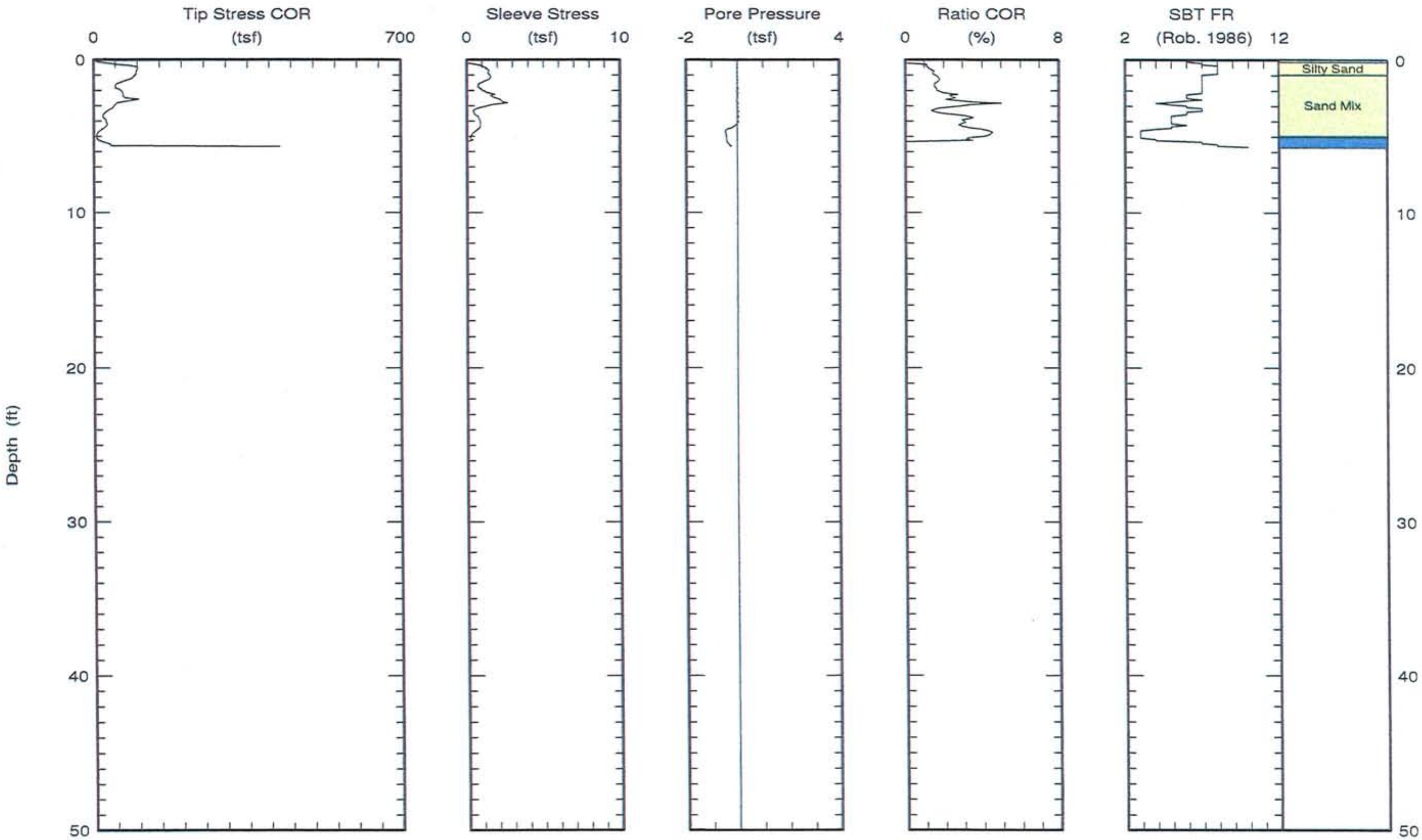
Maximum depth: 75.14 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-2A Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



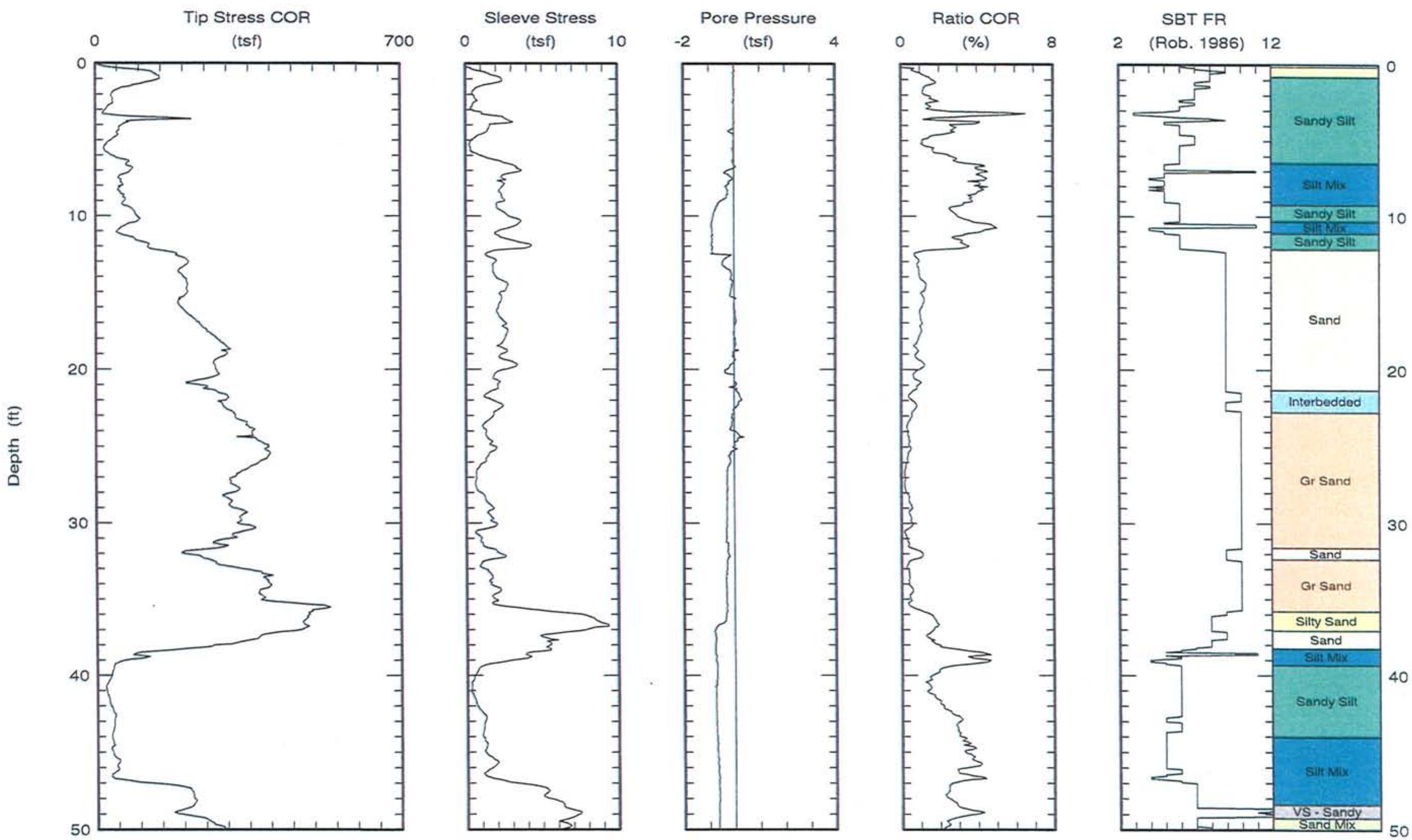
Maximum depth: 7.51 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-2B Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



Maximum depth: 5.70 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-3 Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



Maximum depth: 75.14 (ft)
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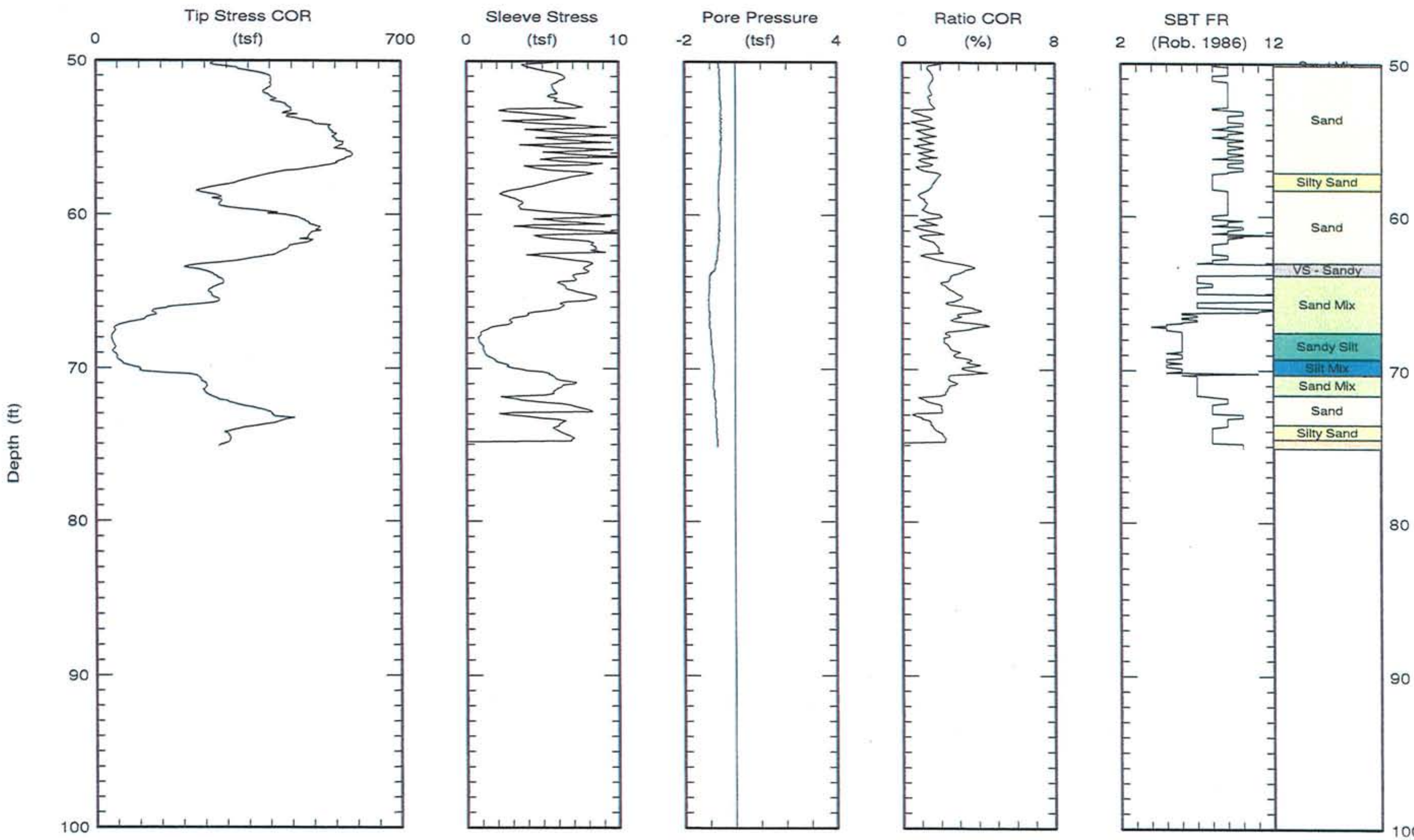
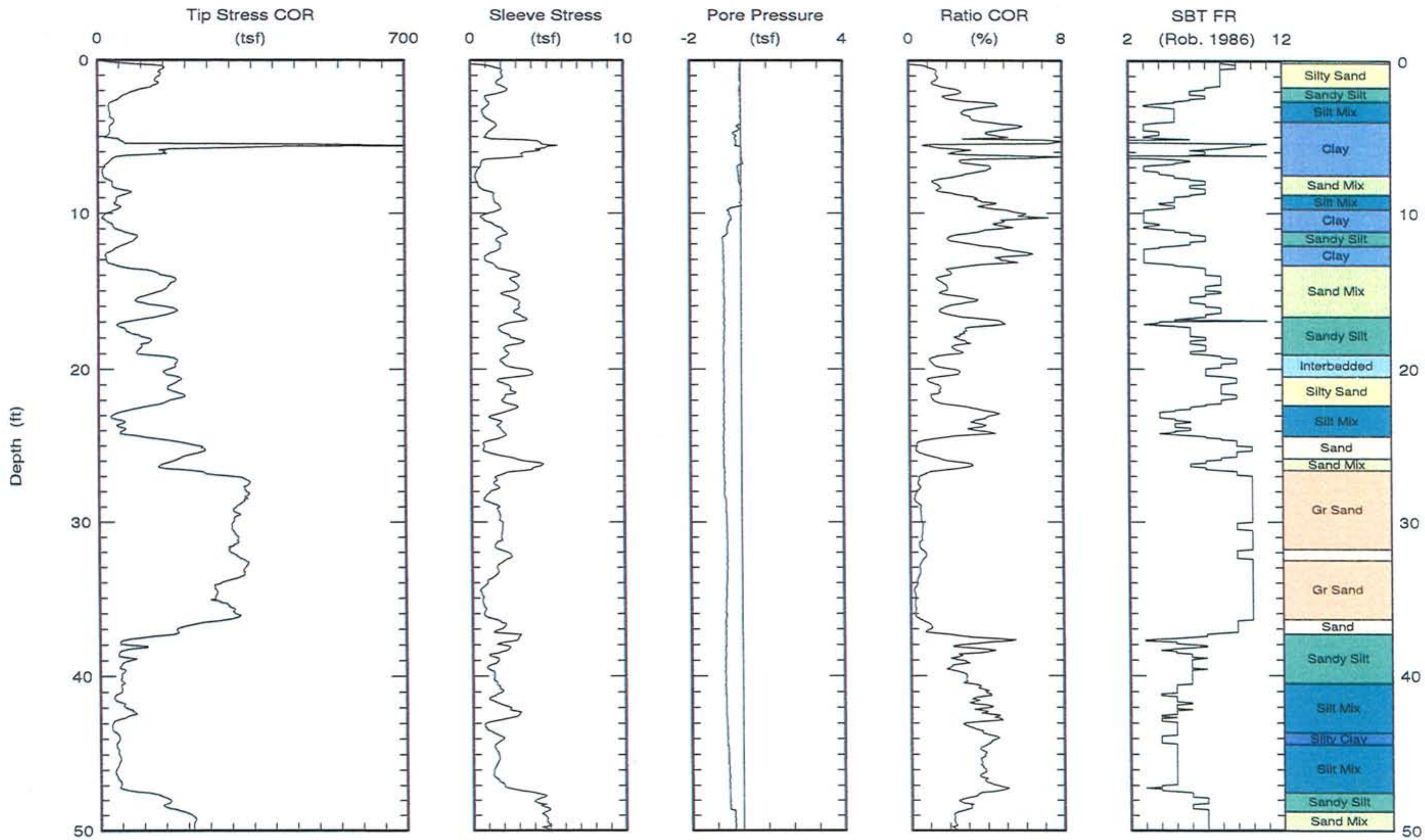
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-3 Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



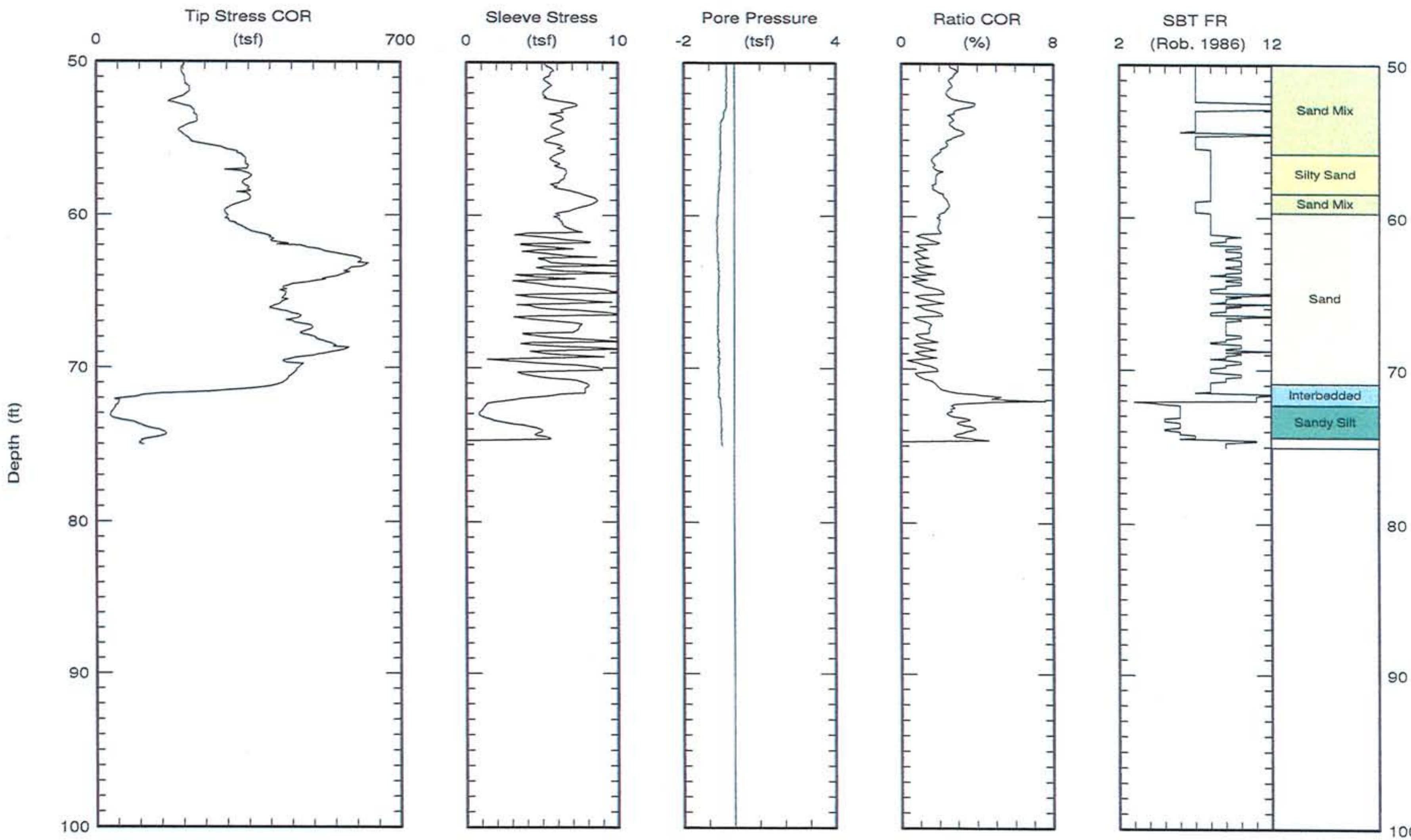
Maximum depth: 75.14 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-4 Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



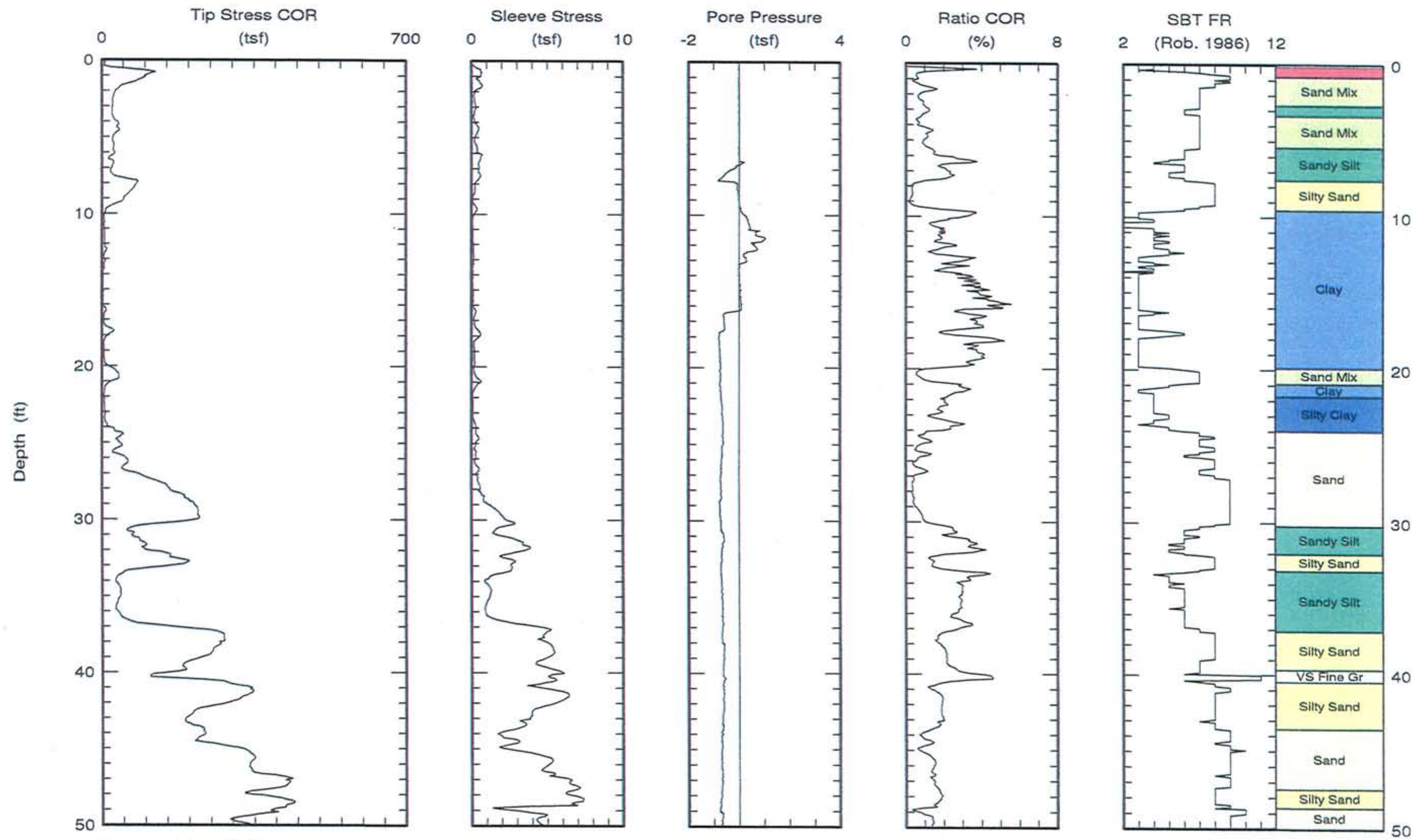
Maximum depth: 75.05 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-4 Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



Maximum depth: 75.05 (ft)

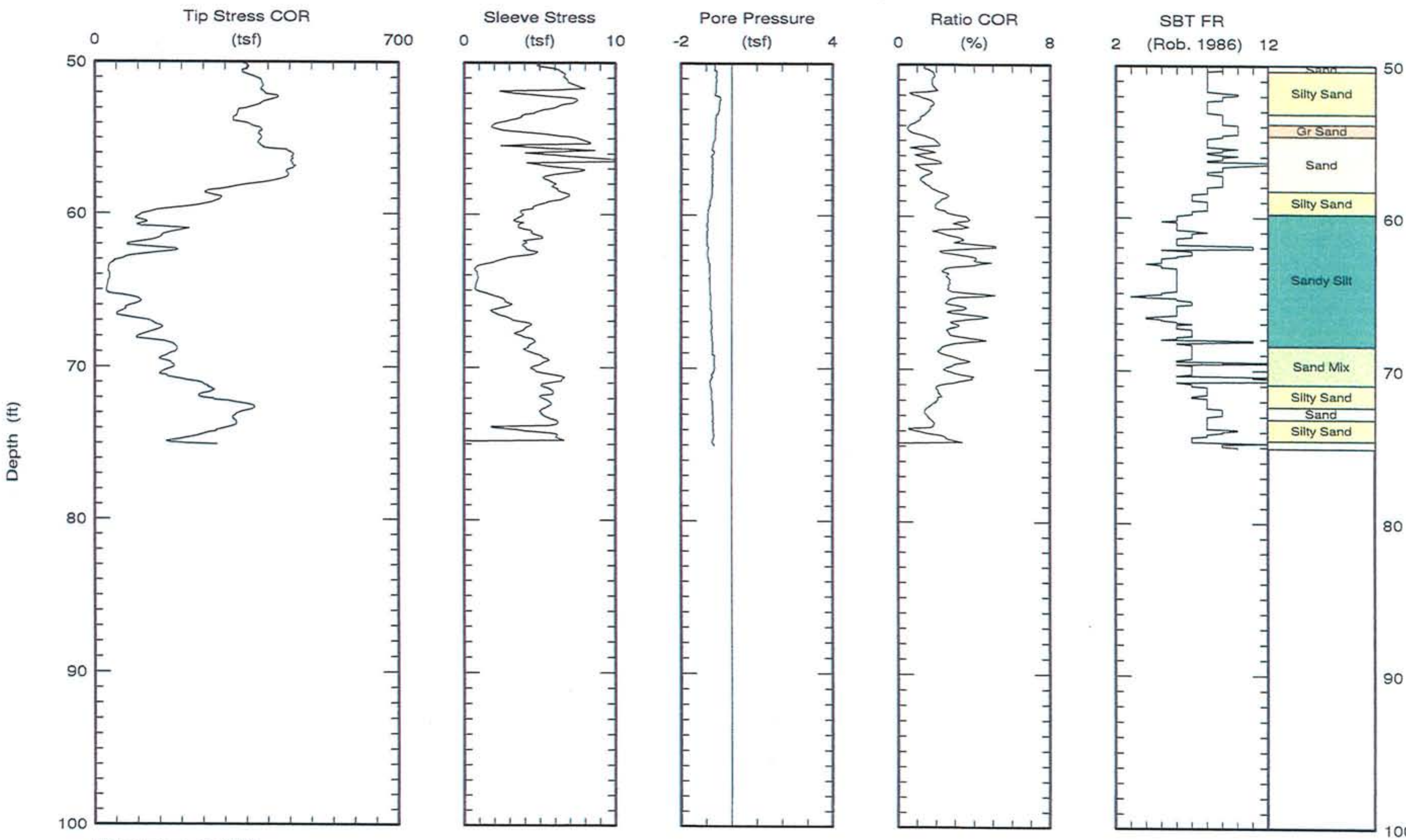
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-5 Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



Maximum depth: 75.09 (ft)

Page 1 of 2

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/May/2011 Test ID: CPT-5 Project: SealBeach
	Customer: D. Scott Magorien Consulting Job Site: Seal Beach DWP Site EIR	



Maximum depth: 75.09 (ft)

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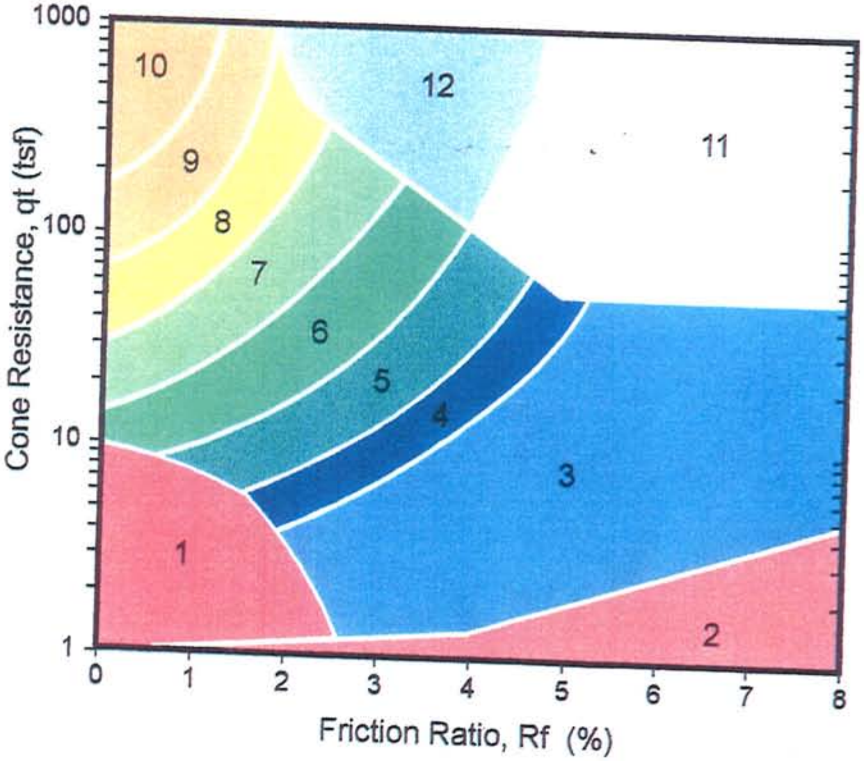
KEHOE TESTING & ENGINEERING

CPT Classification Chart

(after Robertson and Campanella, 1988)



Zone	q_t / N	Soil Behavior Type	UCSCS
1	2	sensitive fine grained	OL-OH
2	1	organic material	Pt-OH
3	1	clay	CH
4	1.5	silty clay to clay	CL-CH
5	2	clayey silt to silty clay	ML-CL
6	2.5	sandy silt to clayey silt	MH-ML
7	3	silty sand to sandy silt	SM-ML
8	4	sand to silty sand	SP-SM
9	5	sand	SP
10	6	gravelly sand to sand	SW-SP
11	1	very stiff fine grained *	CL-MH
12	2	sand to clayey sand *	SP-SC

* overconsolidated or cemented

I PUT FILE: C:\temp\CPT-1.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	53.383	0.193	0.362	8	13	20	9E9
1.500	40.350	0.452	1.120	7	13	20	9E9
2.500	76.833	0.782	1.018	8	18	27	9E9
3.500	170.717	2.137	1.252	8	41	62	9E9
4.500	190.000	1.662	0.875	9	36	54	9E9
5.500	166.050	0.927	0.558	9	32	48	9E9
6.500	171.950	0.967	0.562	9	33	50	9E9
7.500	195.983	0.832	0.424	9	38	57	9E9
8.500	114.967	0.713	0.621	9	22	33	9E9
9.500	43.017	0.568	1.322	7	14	21	9E9
10.500	152.929	1.001	0.655	9	29	44	9E9
11.500	157.700	1.338	0.849	9	30	45	9E9
12.500	98.517	0.988	1.003	8	24	36	9E9
13.500	15.317	0.365	2.378	5	7	11	0.967
14.500	11.550	0.058	0.493	6	5	7	0.729
15.500	26.533	0.292	1.097	7	8	11	9E9
16.500	53.967	1.150	2.138	6	21	28	3.519
17.500	58.767	0.728	1.244	7	19	25	9E9
18.500	80.867	0.555	0.688	8	19	24	9E9
19.500	96.283	0.367	0.382	9	18	22	9E9
20.500	97.700	0.557	0.571	8	23	27	9E9
21.500	119.917	0.905	0.756	9	23	27	9E9
22.500	58.917	1.717	2.924	6	22	25	3.822
23.500	77.783	2.493	3.214	6	30	33	5.075
24.500	155.367	5.915	3.812	12	74	78	9E9
25.500	173.933	5.543	3.191	6	67	69	11.477
26.500	105.150	2.992	2.850	6	40	40	6.888
27.500	48.283	1.845	3.837	5	23	23	3.092
28.500	136.767	1.482	1.085	8	33	32	9E9
29.500	119.417	1.965	1.648	7	38	36	9E9
30.500	81.033	2.110	2.608	6	31	29	5.268
31.500	108.029	2.517	2.330	7	34	31	9E9
32.500	249.450	4.682	1.878	8	60	53	9E9
33.500	259.000	2.320	0.896	9	50	43	9E9
34.500	255.917	1.717	0.671	9	49	42	9E9
35.500	272.800	1.607	0.589	9	52	43	9E9
36.500	237.483	1.312	0.553	9	45	37	9E9
37.500	246.583	2.402	0.975	9	47	38	9E9
38.500	255.217	2.673	1.048	9	49	39	9E9
39.500	253.150	2.768	1.094	9	48	37	9E9
40.500	243.117	2.712	1.116	9	47	36	9E9
41.499	266.629	2.766	1.038	9	51	38	9E9
42.499	265.050	2.465	0.931	9	51	38	9E9
43.499	267.250	2.452	0.918	9	51	37	9E9
44.499	258.767	2.443	0.945	9	50	36	9E9
45.499	124.567	2.418	1.943	7	40	28	9E9
46.499	267.117	1.452	0.544	9	51	35	9E9
47.499	284.250	2.485	0.875	9	54	37	9E9
48.499	268.017	1.865	0.696	9	51	34	9E9
49.499	161.883	1.832	1.132	9	31	21	9E9

IPUT FILE: C:\temp\CPT-1.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	88.417	1.388	1.570	7	28	18	9E9
51.499	62.033	1.078	1.736	7	20	13	9E9
52.499	50.757	1.424	2.808	6	19	12	3.165
53.499	217.383	2.670	1.229	9	42	26	9E9
54.499	389.600	3.288	0.844	10	62	38	9E9
55.499	444.333	3.420	0.770	10	71	43	9E9
56.499	486.283	6.077	1.250	9	93	56	9E9
57.499	503.150	6.360	1.264	9	96	57	9E9
58.499	503.117	4.357	0.866	10	80	47	9E9
59.499	487.783	6.638	1.361	9	93	54	9E9
60.499	519.567	5.478	1.054	9	100	58	9E9
61.499	509.600	5.415	1.063	9	98	56	9E9
62.499	487.500	6.880	1.411	9	93	52	9E9
63.499	446.450	3.430	0.768	10	71	39	9E9
64.499	409.983	2.620	0.639	10	65	36	9E9
65.499	443.133	2.217	0.500	10	71	38	9E9
66.499	417.867	1.717	0.411	10	67	36	9E9
67.499	324.783	1.448	0.446	10	52	28	9E9
68.499	228.500	1.195	0.523	9	44	23	9E9
69.499	166.633	1.492	0.896	9	32	17	9E9
70.499	174.333	1.428	0.820	9	33	17	9E9
71.499	224.067	3.665	1.635	8	54	27	9E9
72.499	134.500	3.237	2.402	7	43	22	9E9
73.499	351.500	4.275	1.217	9	67	34	9E9
74.499	266.450	1.873	0.703	9	51	26	9E9

IPUT FILE: C:\temp\CPT-2.CSV |---

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	78.950	1.312	1.661	7	25	38	9E9
1.500	94.033	2.240	2.382	7	30	45	9E9
2.500	214.150	4.053	1.893	8	51	77	9E9
3.500	110.667	2.602	2.351	7	35	53	9E9
4.500	132.050	1.843	1.396	8	32	48	9E9
5.500	112.850	1.565	1.387	8	27	41	9E9
6.500	51.850	1.072	2.068	6	20	30	3.428
7.500	21.167	0.982	4.663	3	20	30	1.373
8.500	17.083	1.008	5.937	3	16	24	1.097
9.500	21.200	1.392	6.596	3	20	30	1.368
10.500	13.486	0.843	6.297	3	13	20	0.849
11.500	18.850	0.772	4.108	4	12	18	1.205
12.500	29.517	1.148	3.897	4	19	29	1.913
13.500	59.633	2.170	3.642	5	29	44	3.917
14.500	35.350	1.383	3.919	5	17	25	2.294
15.500	55.750	1.898	3.408	5	27	38	3.649
16.500	165.000	3.380	2.049	7	53	72	9E9
17.500	166.817	2.695	1.616	8	40	52	9E9
18.500	124.733	2.105	1.688	7	40	51	9E9
19.500	144.817	2.290	1.582	8	35	43	9E9
20.500	155.457	2.400	1.545	8	37	44	9E9
21.500	144.433	1.952	1.351	8	35	40	9E9
22.500	101.967	1.400	1.374	8	24	27	9E9
23.500	155.850	2.590	1.663	8	37	40	9E9
24.500	189.267	3.090	1.633	8	45	48	9E9
25.500	210.883	3.525	1.672	8	50	52	9E9
26.500	241.783	4.157	1.719	8	58	59	9E9
27.500	227.083	4.402	1.939	8	54	53	9E9
28.500	103.083	3.940	3.824	5	49	47	6.751
29.500	90.600	3.545	3.916	5	43	41	5.912
30.500	36.967	1.082	2.934	6	14	13	2.332
31.500	36.086	1.327	3.688	5	17	15	2.269
32.500	39.600	1.238	3.134	5	19	17	2.500
33.500	79.500	2.067	2.603	6	30	26	5.155
34.500	179.750	3.843	2.139	7	57	48	9E9
35.500	264.183	5.120	1.939	8	63	53	9E9
36.500	202.883	3.363	1.658	8	49	40	9E9
37.500	173.283	2.805	1.619	8	41	33	9E9
38.500	217.483	4.603	2.118	7	69	54	9E9
39.500	254.333	4.970	1.955	8	61	47	9E9
40.500	95.233	3.227	3.392	6	36	27	6.175
41.499	192.171	4.810	2.504	7	61	46	9E9
42.499	245.333	5.885	2.400	7	78	58	9E9
43.499	271.067	5.470	2.019	8	65	47	9E9
44.499	481.733	6.548	1.360	9	92	66	9E9
45.499	593.783	6.527	1.099	9	114	80	9E9
46.499	518.817	6.275	1.210	9	99	69	9E9
47.499	439.200	5.033	1.146	9	84	58	9E9
48.499	434.200	6.713	1.547	9	83	56	9E9
49.499	387.817	7.450	1.922	8	93	62	9E9

IPUT FILE: C:\temp\CPT-2.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	389.750	7.398	1.899	8	93	61	9E9
51.499	351.233	8.525	2.428	7	112	73	9E9
52.499	349.457	7.064	2.022	8	84	54	9E9
53.499	322.883	7.225	2.239	8	77	49	9E9
54.499	305.667	6.188	2.026	8	73	45	9E9
55.499	266.933	3.515	1.318	9	51	31	9E9
56.499	293.900	4.595	1.564	8	70	42	9E9
57.499	299.367	4.595	1.536	8	72	43	9E9
58.499	216.283	5.073	2.347	7	69	41	9E9
59.499	184.017	4.633	2.519	7	59	35	9E9
60.499	181.633	3.772	2.078	7	58	34	9E9
61.499	405.867	6.705	1.653	8	97	56	9E9
62.499	451.129	5.959	1.321	9	86	49	9E9
63.499	598.967	5.453	0.911	10	96	54	9E9
64.499	603.117	7.138	1.184	9	116	64	9E9
65.499	415.483	6.297	1.516	9	80	44	9E9
66.499	349.033	6.738	1.931	8	84	45	9E9
67.499	346.800	6.188	1.785	8	83	44	9E9
68.499	351.800	7.288	2.073	8	84	44	9E9
69.499	349.750	5.308	1.518	9	67	35	9E9
70.499	311.183	5.145	1.654	8	74	38	9E9
71.499	221.783	4.400	1.985	8	53	27	9E9
72.499	342.629	5.360	1.565	8	82	42	9E9
73.499	162.867	5.058	3.109	6	62	31	10.544
74.499	172.067	3.980	2.315	7	55	28	9E9
75.499	319.200	0.000	0.000	10	9E9	9E9	9E9

INPUT FILE: C:\temp\CPT-2A.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	51.883	0.722	1.391	7	17	26	9E9
1.500	72.783	1.608	2.210	7	23	35	9E9
2.500	83.650	2.135	2.553	6	32	48	5.565
3.500	57.150	1.407	2.462	6	22	33	3.795
4.500	64.350	1.895	2.945	6	25	38	4.272
5.500	80.733	2.402	2.976	6	31	47	5.357
6.500	437.883	7.138	1.630	9	84	126	9E9
7.500	609.400	10.420	1.710	9	117	176	9E9

NPUT FILE: C:\temp\CPT-2B.CSV |

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	78.817	1.037	1.315	8	19	29	9E9
1.500	65.417	1.097	1.676	7	21	32	9E9
2.500	67.683	1.853	2.738	6	26	39	4.503
3.500	30.217	0.698	2.311	6	12	18	2.000
4.500	20.283	0.697	3.443	5	10	15	1.330
5.500	7.800	0.260	3.377	3	7	11	0.491

NPUT FILE: C:\temp\CPT-3.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	105.400	1.223	1.160	8	25	38	9E9
1.500	74.167	1.117	1.506	7	24	36	9E9
2.500	35.267	0.560	1.588	7	11	17	9E9
3.500	78.500	1.937	2.468	6	30	45	5.218
4.500	49.783	1.133	2.277	6	19	29	3.300
5.500	25.517	0.412	1.613	6	10	15	1.679
6.500	69.667	2.507	3.598	5	33	50	4.618
7.500	63.467	2.685	4.232	5	30	45	4.199
8.500	59.383	2.383	4.016	5	28	42	3.922
9.500	77.267	2.303	2.986	6	30	45	5.104
10.500	78.371	3.077	3.935	5	37	56	5.170
11.500	91.600	3.017	3.300	6	35	53	6.047
12.500	177.000	2.063	1.166	9	34	51	9E9
13.500	199.050	1.838	0.924	9	38	57	9E9
14.500	208.183	2.542	1.221	9	40	58	9E9
15.500	195.467	2.267	1.160	9	37	51	9E9
16.500	214.233	2.215	1.034	9	41	55	9E9
17.500	254.167	2.653	1.044	9	49	63	9E9
18.500	294.983	2.495	0.846	9	57	71	9E9
19.500	281.017	2.795	0.995	9	54	66	9E9
20.500	255.429	2.097	0.821	9	49	58	9E9
21.500	284.083	1.643	0.578	10	45	51	9E9
22.500	302.433	2.137	0.707	9	58	64	9E9
23.500	343.800	1.363	0.397	10	55	59	9E9
24.500	371.900	1.562	0.420	10	59	62	9E9
25.500	393.333	1.603	0.408	10	63	64	9E9
26.500	344.583	0.823	0.239	10	55	55	9E9
27.500	317.483	0.697	0.219	10	51	49	9E9
28.500	305.167	1.342	0.440	10	49	46	9E9
29.500	333.500	1.660	0.498	10	53	49	9E9
30.500	336.867	1.265	0.376	10	54	49	9E9
31.500	263.571	1.440	0.546	9	50	44	9E9
32.500	274.567	1.517	0.552	9	53	46	9E9
33.500	388.000	1.630	0.420	10	62	53	9E9
34.500	385.317	1.968	0.511	10	62	51	9E9
35.500	479.800	3.610	0.752	10	77	63	9E9
36.500	481.333	8.580	1.783	8	115	92	9E9
37.500	377.867	5.778	1.530	9	72	56	9E9
38.500	157.317	4.710	2.997	6	60	46	10.316
39.500	42.617	1.310	3.085	5	20	15	2.665
40.500	26.450	0.397	1.506	6	10	7	1.585
41.499	27.343	0.494	1.816	6	10	7	1.640
42.499	39.067	1.078	2.769	6	15	11	2.418
43.499	36.950	1.092	2.965	6	14	10	2.271
44.499	35.650	1.232	3.468	5	17	12	2.181
45.499	45.117	1.725	3.838	5	22	15	2.805
46.499	51.367	1.662	3.243	5	25	17	3.220
47.499	205.317	4.958	2.416	7	66	44	9E9
48.499	209.633	6.552	3.127	7	67	44	9E9
49.499	253.767	6.723	2.650	7	81	53	9E9

NPUT FILE: C:\temp\CPT-3.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	317.967	4.942	1.555	8	76	49	9E9
51.499	400.567	6.005	1.500	9	77	49	9E9
52.499	410.514	6.219	1.515	9	79	50	9E9
53.499	456.550	4.555	0.998	9	87	54	9E9
54.499	536.917	6.410	1.194	9	103	63	9E9
55.499	564.750	6.268	1.110	9	108	65	9E9
56.499	558.667	7.417	1.328	9	107	64	9E9
57.499	389.700	6.555	1.682	8	93	55	9E9
58.499	267.917	3.278	1.224	9	51	30	9E9
59.499	323.433	4.160	1.287	9	62	36	9E9
60.499	479.867	6.442	1.343	9	92	52	9E9
61.499	492.517	7.302	1.483	9	94	53	9E9
62.499	403.357	7.199	1.785	8	97	54	9E9
63.499	249.467	7.793	3.126	12	119	65	9E9
64.499	279.300	6.715	2.406	7	89	48	9E9
65.499	257.200	7.442	2.896	7	82	44	9E9
66.499	125.650	4.147	3.305	6	48	26	8.085
67.499	46.467	1.520	3.285	5	22	12	2.802
68.499	40.833	1.015	2.497	6	16	8	2.423
69.499	58.767	1.982	3.382	5	28	14	3.616
70.499	188.317	5.400	2.870	7	60	31	9E9
71.499	255.317	5.452	2.137	8	61	31	9E9
72.499	346.186	5.671	1.639	8	83	42	9E9
73.499	394.833	5.663	1.435	9	76	38	9E9
74.499	305.783	4.435	1.451	9	59	30	9E9
75.499	282.700	0.000	0.000	10	9E9	9E9	9E9

INPUT FILE: C:\temp\CPT-4.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	126.417	1.497	1.184	8	30	45	9E9
1.500	122.883	2.048	1.667	7	39	59	9E9
2.500	46.783	1.288	2.754	6	18	27	3.109
3.500	29.933	0.957	3.196	5	14	21	1.981
4.500	29.283	1.412	4.821	3	28	42	1.934
5.500	219.817	4.137	1.882	8	53	80	9E9
6.500	56.133	1.887	3.361	5	27	41	3.715
7.500	15.650	0.408	2.618	5	7	11	1.009
8.500	49.717	0.935	1.881	7	16	24	9E9
9.500	42.250	1.735	4.110	5	20	30	2.775
10.500	24.500	1.270	5.205	3	23	35	1.583
11.500	75.267	2.028	2.700	6	29	44	4.961
12.500	29.583	1.430	4.853	3	28	42	1.913
13.500	78.300	2.012	2.572	6	30	45	5.158
14.500	165.100	2.915	1.767	8	40	58	9E9
15.500	108.767	2.715	2.500	7	35	49	9E9
16.500	145.533	3.207	2.205	7	46	62	9E9
17.500	65.917	2.202	3.347	6	25	33	4.313
18.500	103.467	2.737	2.648	6	40	51	6.813
19.500	159.583	2.235	1.402	8	38	46	9E9
20.500	168.157	2.943	1.751	8	40	47	9E9
21.500	174.767	2.423	1.388	8	42	48	9E9
22.500	88.250	2.367	2.685	6	34	38	5.784
23.500	49.433	1.740	3.531	5	24	26	3.188
24.500	129.600	1.593	1.231	8	31	33	9E9
25.500	212.700	1.693	0.797	9	41	42	9E9
26.500	189.883	3.520	1.855	8	45	45	9E9
27.500	335.517	1.617	0.482	10	54	53	9E9
28.500	331.517	1.123	0.339	10	53	51	9E9
29.500	311.517	1.807	0.580	10	50	47	9E9
30.500	305.533	2.003	0.656	10	49	45	9E9
31.500	306.886	1.821	0.594	10	49	44	9E9
32.500	328.183	2.148	0.655	10	52	45	9E9
33.500	315.800	1.447	0.458	10	50	43	9E9
34.500	266.317	0.723	0.272	10	42	35	9E9
35.500	289.550	0.780	0.269	10	46	38	9E9
36.500	256.417	1.615	0.630	9	49	39	9E9
37.500	115.033	2.385	2.076	7	37	29	9E9
38.500	64.883	1.870	2.887	6	25	19	4.157
39.500	52.233	1.322	2.537	6	20	15	3.308
40.500	49.950	1.608	3.230	5	24	18	3.151
41.499	45.043	1.693	3.770	5	22	16	2.820
42.499	56.567	2.250	3.986	5	27	20	3.586
43.499	34.767	1.315	3.795	5	17	12	2.128
44.499	40.317	1.583	3.939	5	19	13	2.494
45.499	42.317	1.583	3.750	5	20	14	2.624
46.499	40.267	1.567	3.902	5	19	13	2.482
47.499	98.700	3.573	3.624	6	38	26	6.375
48.499	165.683	4.512	2.724	7	53	35	9E9
49.499	212.067	4.793	2.261	7	68	45	9E9

INPUT FILE: C:\temp\CPT-4.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	199.633	5.432	2.722	7	64	42	9E9
51.499	207.733	5.322	2.563	7	66	42	9E9
52.499	195.643	6.034	3.085	7	62	39	9E9
53.499	227.733	6.113	2.685	7	73	46	9E9
54.499	200.517	5.905	2.946	7	64	40	9E9
55.499	272.467	5.948	2.184	8	65	40	9E9
56.499	343.383	5.897	1.718	8	82	49	9E9
57.499	340.600	6.405	1.881	8	82	49	9E9
58.499	348.817	7.127	2.044	8	83	49	9E9
59.499	315.067	7.513	2.386	7	101	59	9E9
60.499	316.650	6.333	2.001	8	76	44	9E9
61.499	392.633	5.955	1.517	9	75	43	9E9
62.499	545.171	5.143	0.944	10	87	49	9E9
63.499	589.283	7.023	1.192	9	113	63	9E9
64.499	468.717	6.232	1.330	9	90	49	9E9
65.499	428.733	6.548	1.528	9	82	44	9E9
66.499	440.883	6.382	1.448	9	84	45	9E9
67.499	483.283	6.453	1.336	9	93	49	9E9
68.499	543.350	6.632	1.221	9	104	55	9E9
69.499	474.250	5.353	1.129	9	91	47	9E9
70.499	450.100	6.118	1.360	9	86	44	9E9
71.499	293.850	7.448	2.536	7	94	48	9E9
72.499	48.014	1.696	3.540	5	23	12	2.892
73.499	78.533	2.755	3.512	6	30	15	4.924
74.499	132.583	3.388	2.558	7	42	21	9E9

INPUT FILE: C:\temp\CPT-5.CSV

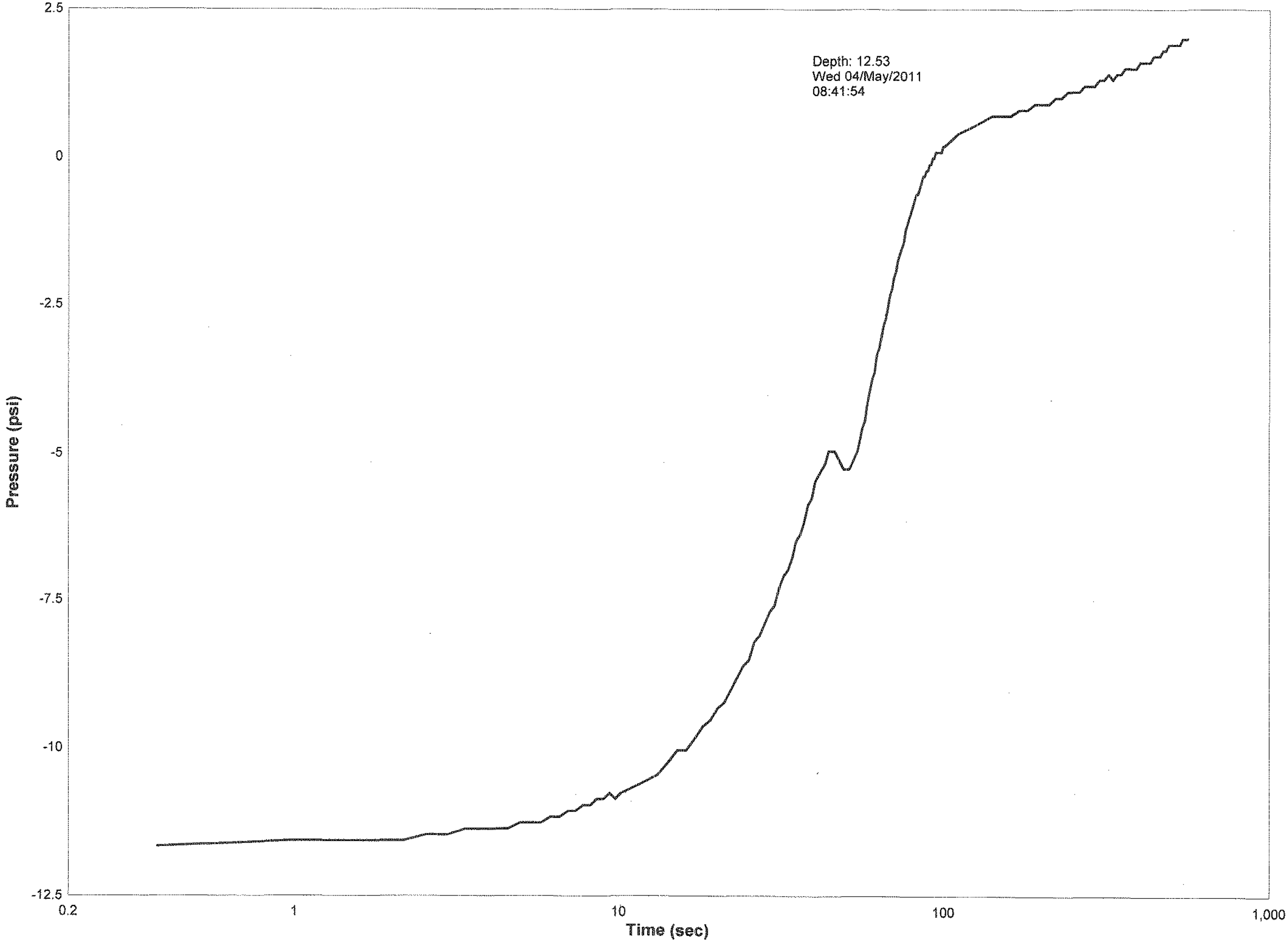
Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	60.767	0.432	0.710	8	15	23	9E9
1.500	61.383	0.517	0.842	8	15	23	9E9
2.500	26.317	0.235	0.893	7	8	12	9E9
3.500	25.800	0.217	0.840	7	8	12	9E9
4.500	35.333	0.397	1.123	7	11	17	9E9
5.500	26.833	0.325	1.211	6	10	15	1.766
6.500	24.267	0.583	2.404	6	9	14	1.591
7.500	39.800	0.495	1.247	7	13	20	9E9
8.500	67.900	0.222	0.326	8	16	24	9E9
9.500	28.267	0.237	0.837	7	9	14	9E9
10.500	5.800	0.116	1.976	4	4	6	0.348
11.500	6.167	0.127	2.005	4	4	6	0.374
12.500	7.483	0.158	2.097	4	5	8	0.452
13.500	5.250	0.128	2.437	3	5	8	0.296
14.500	3.300	0.113	3.434	3	3	4	0.161
15.500	3.283	0.142	4.315	3	3	4	0.156
16.500	6.333	0.223	3.564	3	6	8	0.351
17.500	14.017	0.373	2.689	5	7	9	0.855
18.500	6.633	0.270	4.175	3	6	8	0.356
19.500	6.000	0.213	3.657	3	6	8	0.310
20.500	30.743	0.326	1.064	7	10	12	9E9
21.500	7.733	0.210	2.763	4	5	6	0.420
22.500	6.167	0.110	1.838	4	4	5	0.308
23.500	9.300	0.203	2.218	4	6	7	0.516
24.500	37.367	0.373	1.003	7	12	13	9E9
25.500	39.117	0.275	0.705	7	12	13	9E9
26.500	56.233	0.363	0.648	8	13	14	9E9
27.500	121.817	0.473	0.389	9	23	23	9E9
28.500	183.483	0.750	0.409	9	35	35	9E9
29.500	218.933	1.722	0.787	9	42	41	9E9
30.500	115.517	2.058	1.784	7	37	35	9E9
31.500	88.643	3.140	3.548	6	34	32	5.772
32.500	169.217	2.502	1.480	8	40	36	9E9
33.500	56.050	1.770	3.167	6	21	19	3.590
34.500	40.433	1.193	2.960	6	15	13	2.548
35.500	34.767	1.003	2.898	6	13	11	2.163
36.500	64.717	1.953	3.026	6	25	21	4.155
37.500	258.200	4.825	1.870	8	62	51	9E9
38.500	258.267	5.303	2.054	8	62	50	9E9
39.500	198.317	4.825	2.434	7	63	50	9E9
40.500	208.117	5.102	2.453	7	66	51	9E9
41.499	327.286	5.847	1.787	8	78	60	9E9
42.499	219.150	4.187	1.912	8	52	39	9E9
43.499	215.050	2.973	1.383	8	51	38	9E9
44.499	248.883	2.433	0.978	9	48	35	9E9
45.499	342.717	4.568	1.333	9	66	47	9E9
46.499	367.317	5.228	1.424	9	70	50	9E9
47.499	396.250	6.647	1.678	8	95	66	9E9
48.499	413.000	6.078	1.472	9	79	54	9E9
49.499	340.500	4.267	1.253	9	65	44	9E9

INPUT FILE: C:\temp\CPT-5.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	348.700	5.865	1.683	8	83	55	9E9
51.499	384.350	6.397	1.665	8	92	60	9E9
52.499	392.100	6.119	1.561	9	75	49	9E9
53.499	329.167	3.715	1.129	9	63	40	9E9
54.499	377.967	4.105	1.086	9	72	45	9E9
55.499	401.733	5.852	1.457	9	77	48	9E9
56.499	456.017	6.797	1.491	9	87	53	9E9
57.499	433.650	6.243	1.440	9	83	50	9E9
58.499	296.083	6.410	2.166	8	71	43	9E9
59.499	220.400	4.860	2.207	7	70	41	9E9
60.499	111.583	3.668	3.292	6	43	25	7.180
61.499	158.467	4.527	2.860	7	51	30	9E9
62.499	113.329	3.841	3.396	6	43	25	7.286
63.499	35.217	1.077	3.069	5	17	10	2.079
64.499	31.117	0.840	2.714	6	12	7	1.799
65.499	75.233	2.323	3.094	6	29	16	4.737
66.499	72.933	2.462	3.382	6	28	15	4.579
67.499	135.350	3.958	2.928	6	52	28	8.735
68.499	158.550	4.345	2.743	7	51	27	9E9
69.499	169.217	5.085	3.007	7	54	29	9E9
70.499	175.300	5.617	3.206	6	67	35	11.389
71.499	256.983	5.512	2.146	8	62	32	9E9
72.499	326.829	5.369	1.643	8	78	40	9E9
73.499	319.950	4.955	1.549	8	77	39	9E9
74.499	226.733	3.773	1.665	8	54	27	9E9

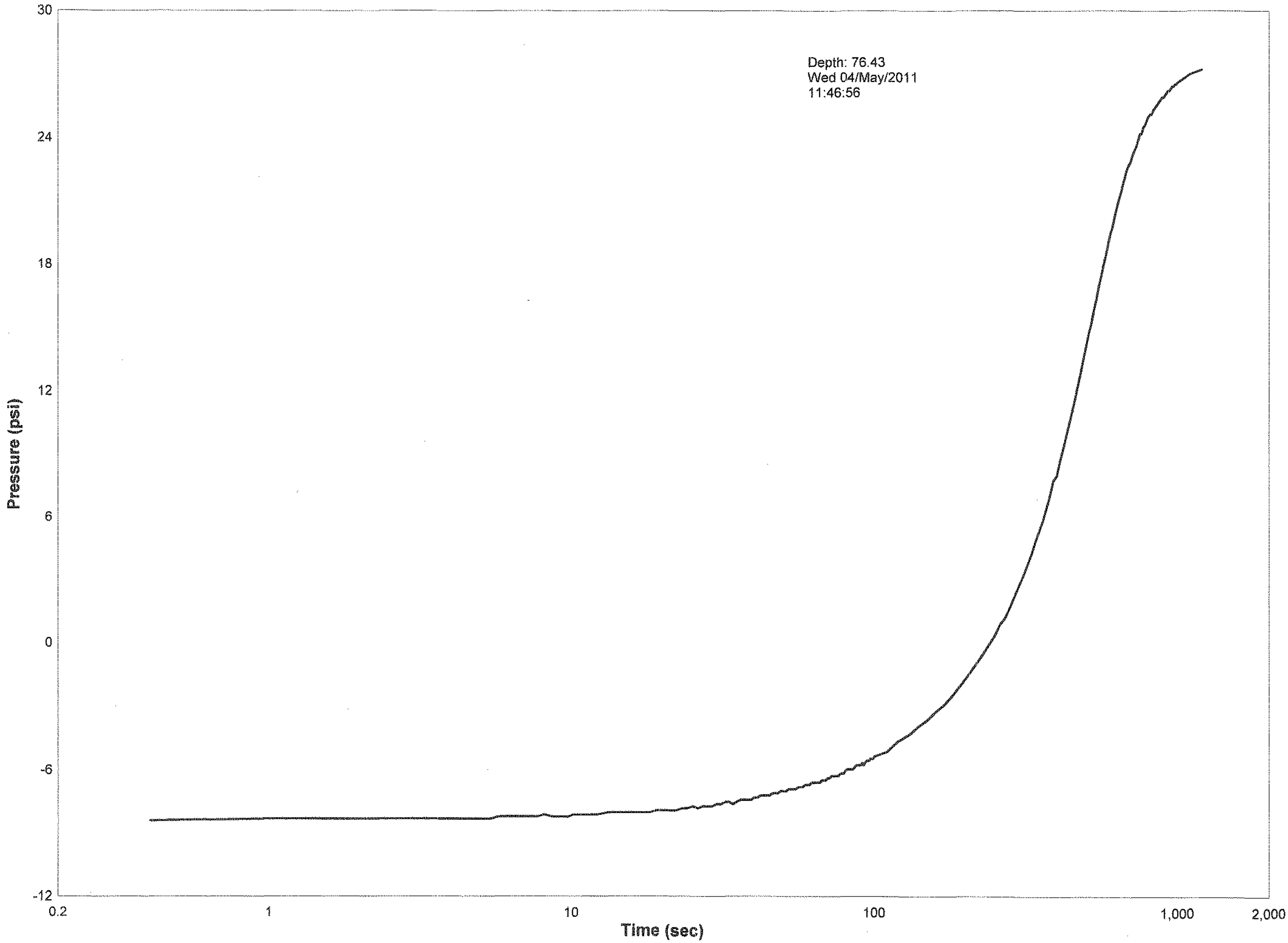
D. Scott Magorien Consulting CPT-3

Kehoe Testing



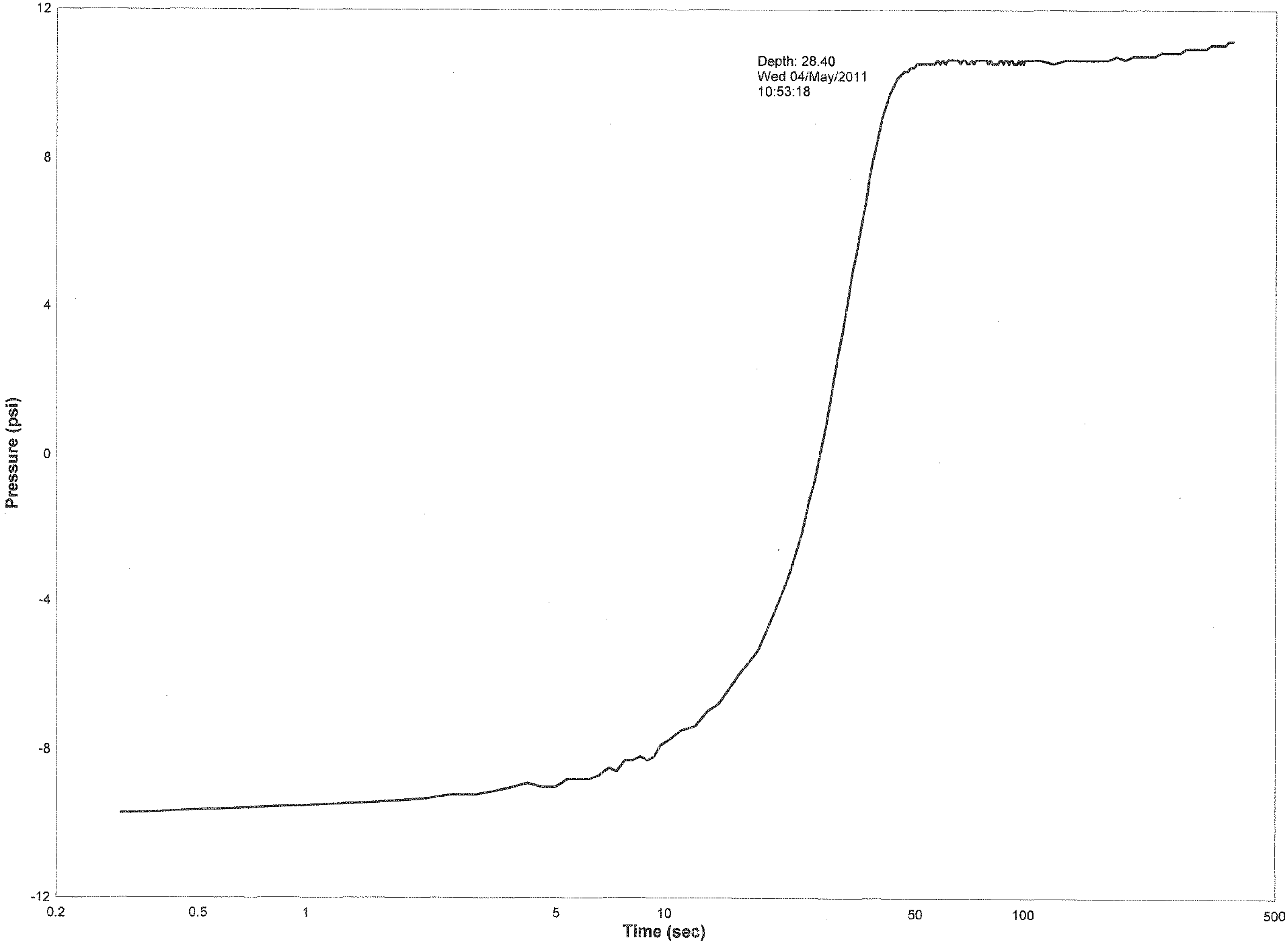
D. [illegible]ott [illegible]gor[illegible] Co[illegible]ulti[illegible]g C[illegible]-2

Kehoe Testing



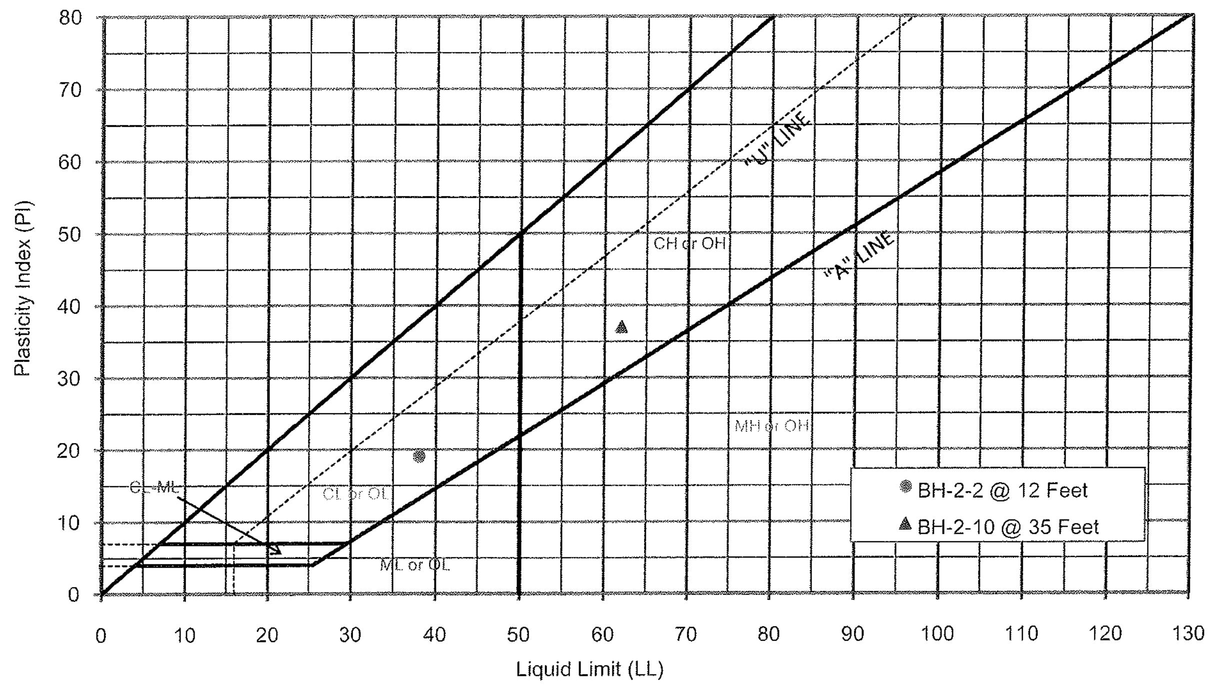
D. Scott Magorian Consulting C-5

Kehoe Testing



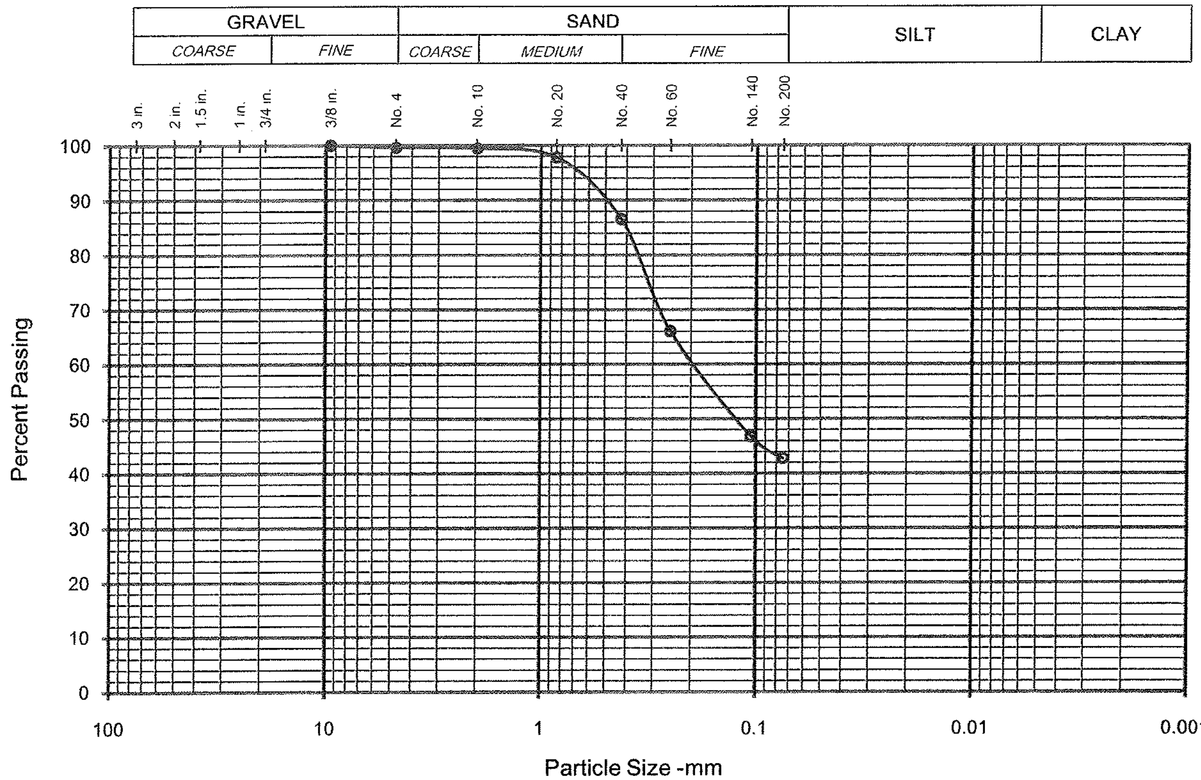
APPENDIX B

Laboratory Test Data



Sample Identification	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Soil Classification
BH-2-2 @ 12 Feet	38	19	19	Lean Clay with Sand (CL)
BH-2-10 @ 35 Feet	62	25	37	Fat Clay (CH)

AMEC Geomatrix	PLASTICITY INDEX (PI) SEAL BEACH DWP SPECIFIC PLAN EIR Seal Beach, California	Project No. NB11161340

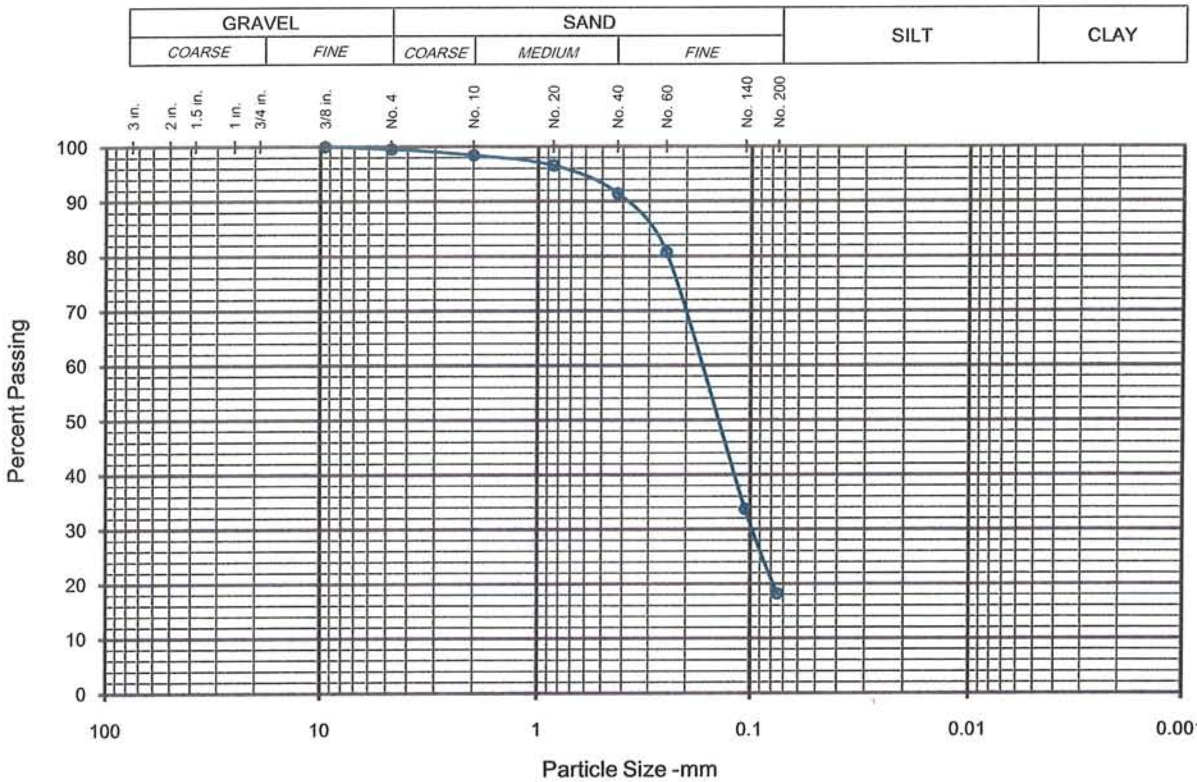


Sieve No.	Opening (mm)	Percent Finer	Sieve No.	Opening (mm)	Percent Finer	Particle Size (mm)	Percent Finer
3 in.	75.0		No. 4	2.80	99.6		
2 in.	50.0		No. 10	2.00	99.5		
1.5 in.	38.1		No. 20	1.40	97.8		
1 in.	25.0		No. 40	1.00	86.6		
3/4 in.	19.0		No. 60	0.71	66.1		
3/8 in.	9.5	100.0	No. 140	0.50	46.8		
			No. 200	0.335	42.8		
% GRAVEL			% SAND			% FINES	
Coarse	Fine		Coarse	Medium	Fine	Silt and Clay	
0.0	0.4		0.2	12.9	43.8	42.8	

<u>Boring No.</u>
BH-2
<u>Sample Depth</u>
21 Feet
<u>Soil Description</u>
Clayey Sand

<u>Group Symbol</u>
SC

AMEC Geomatrix	GRAIN SIZE DISTRIBUTION CURVE SEAL BEACH DWP SPECIFIC PLAN EIR Seal Beach, California	Project No. NB11161340

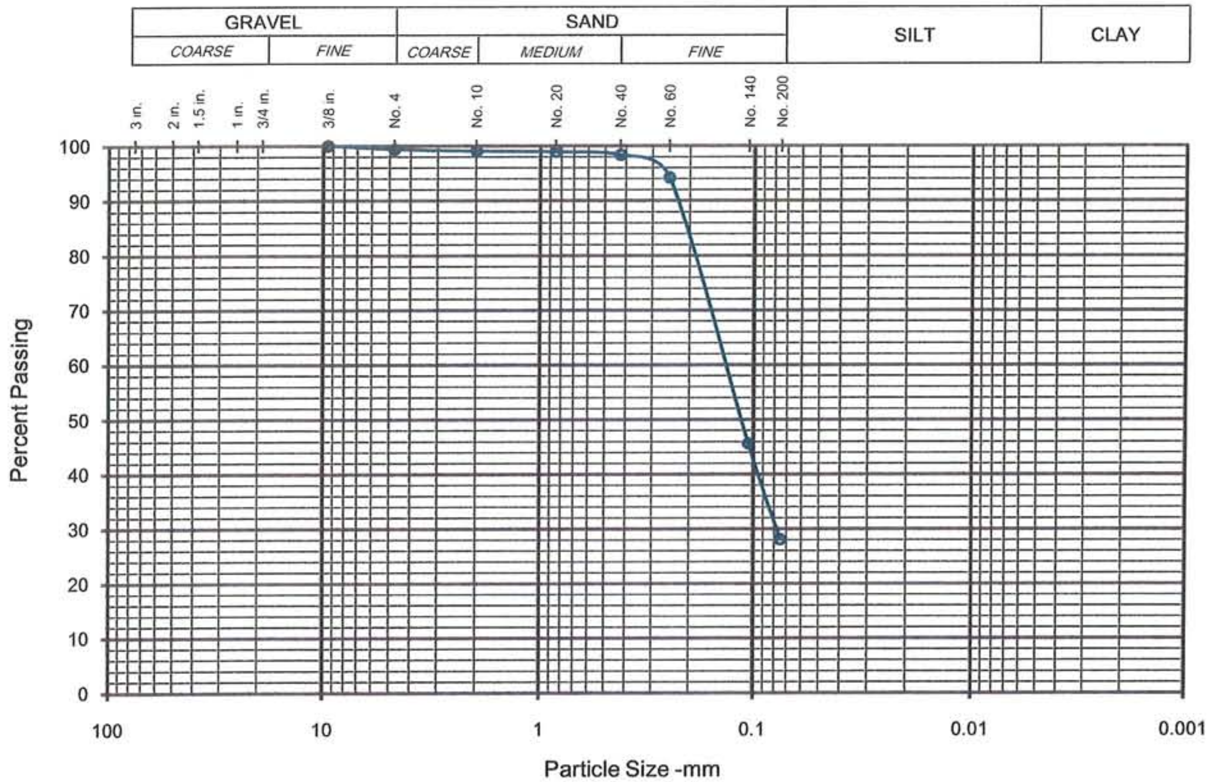


Sieve No.	Opening (mm)	Percent Finer	Sieve No.	Opening (mm)	Percent Finer	Particle Size (mm)	Percent Finer
3 in.	75.0		No. 4	2.80	99.5		
2 in.	50.0		No. 10	2.00	98.4		
1.5 in.	38.1		No. 20	1.40	96.6		
1 in.	25.0		No. 40	1.00	91.5		
3/4 in.	19.0		No. 60	0.71	80.7		
3/8 in.	9.5	100.0	No. 140	0.50	33.7		
			No. 200	0.335	18.2		
% GRAVEL			% SAND			% FINES	
Coarse	Fine		Coarse	Medium	Fine	Silt and Clay	
0.0	0.5		1.1	7.0	73.3	18.2	

Boring No.
BH-2
Sample Depth
27 Feet
Soil Description
Silty Sand

Group Symbol
SM

AMEC Geomatrix	GRAIN SIZE DISTRIBUTION CURVE SEAL BEACH DWP SPECIFIC PLAN EIR Seal Beach, California	Project No. NB11161340



Sieve No.	Opening (mm)	Percent Finer	Sieve No.	Opening (mm)	Percent Finer	Particle Size (mm)	Percent Finer
3 in.	75.0		No. 4	2.80	99.4		
2 in.	50.0		No. 10	2.00	99.1		
1.5 in.	38.1		No. 20	1.40	98.9		
1 in.	25.0		No. 40	1.00	98.4		
3/4 in.	19.0		No. 60	0.71	94.2		
3/8 in.	9.5	100.0	No. 140	0.50	45.6		
			No. 200	0.335	28.1		

% GRAVEL		% SAND			% FINES
Coarse	Fine	Coarse	Medium	Fine	Silt and Clay
0.0	0.6	0.3	0.7	70.3	28.1

Boring No.
BH-1
Sample Depth
15 Feet
Soil Description
Silty Sand

Group Symbol
SM

AMEC Geomatrix	GRAIN SIZE DISTRIBUTION CURVE SEAL BEACH DWP SPECIFIC PLAN EIR Seal Beach, California	Project No. NB11161340

AMEC Geomatrix

MATERIAL IN SOILS FINER THAN No. 200 SIEVE
(ASTM-D1140)

Project Name:	Seal Beach DWP Specific Plan EIR	*Project No.:*	NB11161340
Date:	5/24-5/25/2011	*Tested By:*	VC, LT

Boring No.	BH-1		BH-2		BH-2		
Sample No.	16		17		19		
Sample Depth (Ft)	45		65		75		
Tare No.:	13		18		14		
Total Dry Weight and Tare (g):	316.75		241.96		280.80		
Tare Weight (g):	98.36		90.52		98.39		
Total Dry Weight of Sample (g):	218.39		151.44		182.41		
Dry Weight of Soil Retained on No. 200 Sieve (g):	198.88		23.75		60.60		
Percentage of Material Finer Than No. 200 (75 mm) Sieve (%):	8.9		84.3		66.8		
Soil Description	Poorly Graded Sand with Silt (SP-SM)		Lean Clay with Sand (CL)		Sandy Silt (ML)		

AMEC Geomatrix

MOISTURE CONTENT AND DRY DENSITY TEST

Project Name: Seal Beach DWP Specific Plan EIR *Project No.:* NB11161340
Date: 5/24/2011 *Tested By:* VC, LT

Boring No.	Sample No.	Depth (ft)	Wet Soil and Rings (gr)	Number of Rings	Weight of Rings (gr)	Volume of Sample (cc)	Wet Density (pcf)	Wet Soil and Tare (gr)	Dry Soil and Tare (gr)	Tare No.	Tare Weight (gr)	Moisture Content (%)	Dry Density (pcf)	Soil Description
BH-1	10	28.5	789.48	4	176.86	300.50	127.3	320.15	269.11	MC-60	50.84	23.4	103.1	Poorly Graded Sand with Silt (SP-SM)
BH-2	2	12	569.56	3	132.65	225.38	121.0	153.10	128.94	MC-53	50.42	30.8	92.5	Lean Clay with Sand (CL)
BH-2	10	35	746.48	4	176.86	300.50	118.3	163.88	135.73	MC-17	51.40	33.4	88.7	Fat Clay (CH)

Ring I.D. (in.) = 2.416 Ring Height (in.) = 1.000
Average weight of 1 ring (gr.): 44.22

APPENDIX C

Seismically-Induced Settlement Calculations

Reference: Idriss and Boulanger (2008), Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, Monograph-12

Project: Seal Beach
Project Number: NB11161340

PGA = 0.46 g
M = 7.03
GWT = 9 feet at time of drilling
GWT = 9 feet during earthquake
Total Unit Weight Above Water 120
Total Unit Weight below Water 120
Borehole diameter 115 mm
Sampler liner correction (yes/no) no
Energy Efficient 60 %

C_E = 1
C_B = 1
MSF = 1.13

Boring BH-1

Depth of top of layer (feet)	Soil Properties: Soil type	Measured blow count N_m	Non Standard Sampler	Fines content (%)	Compute CRR: Total unit weight γ_t (pcf)	Vertical stress at time of drilling: σ_v (psf)	σ_v' (psf)	r_d	C_R	C_S	N_{60}	C_N	$(N_1)_{60}$	$\Delta(N_1)_{60}$	$(N_1)_{60,CS}$	$(CRR)_{M=7.5}$	Compute CSR: K_σ	Total unit weight γ_t (pcf)	Vertical stress during earthquake: σ_v (psf)	σ_v' (psf)	$(CSR)_{M=7.5}$	Factor of Safety	Saturated Settlement (Tokimatsu and Seed 1987): Volumetric Strain (%)	Layer Settlement (in)	Cumulative Settlement (in)
0.01	ml	7	1.00	60	120.0	1.20	1	1.01	0.75	1.00	5.25	1.70	9	6	15	0.15	1.10	120.0	1.20	1.20	0.24	**5.00**	0.00	0.00	6.4
2.5	sc	21	1.00	30	120.0	300.00	300	1.00	0.75	1.00	15.75	1.70	27	5	32	0.66	1.10	120.0	300.00	300.00	0.24	**5.00**	0.00	0.00	6.4
4	sp	21	1.00	5	120.0	480.00	480	1.00	0.75	1.00	15.75	1.70	27	0	27	0.34	1.10	120.0	480.00	480.00	0.24	**5.00**	0.00	0.00	6.4
5.5	sm	31	1.00	15	120.0	660.00	660	0.99	0.80	1.00	24.80	1.44	36	3	39	2.89	1.10	120.0	660.00	660.00	0.24	**5.00**	0.00	0.00	6.4
9	sp	22	1.00	5	120.0	1080.00	1080	0.98	0.85	1.00	18.70	1.29	24	0	24	0.27	1.10	120.0	1080.00	1080.00	0.24	**1.14**	0.00	0.00	6.4
11.5	sp	11	1.00	5	120.0	1380.00	1224	0.97	0.85	1.00	9.35	1.29	12	0	12	0.13	1.05	120.0	1380.00	1224.00	0.27	**0.48**	2.33	0.84	6.4
14.5	sm	17	1.00	28.1	120.0	1740.00	1397	0.96	0.85	1.00	14.45	1.18	17	5	22	0.24	1.04	120.0	1740.00	1396.80	0.30	**0.79**	1.27	0.38	5.5
17	sm	7	1.00	35	120.0	2040.00	1541	0.95	0.95	1.00	6.65	1.16	8	6	13	0.14	1.02	120.0	2040.00	1540.80	0.32	**0.44**	2.17	0.91	5.1
20.5	sp	24	1.00	5	120.0	2460.00	1742	0.93	0.95	1.00	22.80	1.06	24	0	24	0.27	1.02	120.0	2460.00	1742.40	0.34	**0.80**	1.23	1.03	4.2
27.5	sp-sm	21	1.00	10	120.0	3300.00	2146	0.89	0.95	1.00	19.95	0.97	19	1	20	0.21	0.99	120.0	3300.00	2145.60	0.37	**0.58**	1.54	0.92	3.2
32.5	sp	29	1.00	5	120.0	3900.00	2434	0.87	1.00	1.00	29.00	0.93	27	0	27	0.34	0.97	120.0	3900.00	2433.60	0.38	**0.90**	1.07	0.32	2.3
35	sp	32	1.00	5	120.0	4200.00	2578	0.85	1.00	1.00	32.00	0.91	29	0	29	0.44	0.95	120.0	4200.00	2577.60	0.39	**1.13**	0.00	0.00	2.0
41.5	sp	27	1.00	5	120.0	4980.00	2952	0.82	1.00	1.00	27.00	0.85	23	0	23	0.25	0.94	120.0	4980.00	2952.00	0.39	**0.64**	1.36	0.41	2.0
44	sp-sm	31	1.00	8.9	120.0	5280.00	3096	0.80	1.00	1.00	31.00	0.84	26	1	27	0.34	0.93	120.0	5280.00	3096.00	0.39	**0.87**	1.07	0.90	1.5
51	sc	11	1.00	30	120.0	6120.00	3499	0.77	1.00	1.00	11.00	0.73	8	5	13	0.14	0.95	120.0	6120.00	3499.20	0.37	**0.38**	2.17	0.65	0.7
53.5	sp	46	1.00	5	120.0	6420.00	3643	0.75	1.00	1.00	46.00	0.83	38	0	38	2.41	0.82	120.0	6420.00	3643.20	0.43	**5.00**	0.00	0.00	0.0
58	sp	63	1.00	5	120.0	6960.00	3902	0.73	1.00	1.00	63.00	0.84	53	0	53	5593.41	0.80	120.0	6960.00	3902.40	0.43	**5.00**	0.00	0.00	0.0
68	sp	59	1.00	5	120.0	8160.00	4478	0.68	1.00	1.00	59.00	0.81	48	0	48	137.35	0.76	120.0	8160.00	4478.40	0.43	**5.00**	0.00	0.00	0.0
76.5																									

Reference: Idriss and Boulanger (2008), Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, Monograph-12

Project:	Seal Beach
Project Number:	NB11161340
PGA =	0.46 g
M =	7.03
GWT =	9 feet at time of drilling
GWT =	9 feet during earthquake
Total Unit Weight Above Water	120
Total Unit Weight below Water	120
Borehole diameter	115 mm
Sampler liner correction (yes/no)	no
Energy Efficient	60 %
C_E =	1
C_B =	1
MSF =	1.13

Boring BH-2

Depth of top of layer (feet)	Soil Properties: Soil type	Measured blow count N_m	Non Standard Sampler	Fines content (%)	Compute CRR: Total unit weight γ_t (pcf)	Vertical stress at time of drilling σ_v (psf)	σ_v' (psf)	r_d	C_R	C_S	N_{60}	C_N	$(N_1)_{60}$	$\Delta(N_1)_{60}$	$(N_1)_{60,CS}$	$(CRR)_{M=7.5}$	Compute CSR: K_σ	Total unit weight γ_t (pcf)	Vertical stress during earthquake σ_v (psf)	σ_v' (psf)	$(CSR)_{M=7.5}$	Factor of Safety	SATURATED SETTLEMENT (Tokimatsu and Seed 1987): Volumetric Strain (%)	Layer Settlement (in)	Cumulative Settlement (in)
0.01	sc/sm	5	1.00	35	120.0	1.20	1	1.01	0.75	1.00	3.75	1.70	6	6	12	0.13	1.10	120.0	1.20	1.20	0.24	**5.00**	0.00	0.00	6.2
7.5	cl	2	1.00	95	120.0	900.00	900	0.98	0.80	1.00	1.60	1.69	3	6	8	0.11	1.05	120.0	900.00	900.00	0.25	**5.00**	0.00	0.00	6.2
9	cl	2	0.65	95	120.0	1080.00	1080	0.98	0.85	1.00	1.11	1.52	2	6	7	0.10	1.04	120.0	1080.00	1080.00	0.25	**5.00**	0.00	0.00	6.2
14.5	ml	4	1.00	55	120.0	1740.00	1397	0.96	0.85	1.00	3.40	1.25	4	6	10	0.12	1.03	120.0	1740.00	1396.80	0.31	**0.38**	2.67	0.96	6.2
17.5	sc/sm	10	1.00	40	120.0	2100.00	1570	0.94	0.95	1.00	9.50	1.14	11	6	16	0.17	1.02	120.0	2100.00	1569.60	0.33	**0.52**	1.83	0.77	5.2
21	sc	1	1.00	42.8	120.0	2520.00	1771	0.93	0.95	1.00	0.95	1.09	1	6	7	0.10	1.01	120.0	2520.00	1771.20	0.35	**0.28**	3.50	1.05	4.4
23.5	sm	25	0.65	40	120.0	2820.00	1915	0.91	0.95	1.00	15.44	1.02	16	6	21	0.22	1.00	120.0	2820.00	1915.20	0.35	**0.63**	1.46	0.53	3.4
26.5	sm	8	1.00	18.2	120.0	3180.00	2088	0.90	0.95	1.00	7.60	0.98	7	4	12	0.13	1.00	120.0	3180.00	2088.00	0.36	**0.36**	2.33	0.84	2.9
29.5	sp	22	1.00	5	120.0	3540.00	2261	0.88	1.00	1.00	22.00	0.95	21	0	21	0.22	0.98	120.0	3540.00	2260.80	0.37	**0.58**	1.46	0.70	2.0
33.5	ch	27	0.65	95	120.0	4020.00	2491	0.86	1.00	1.00	17.55	0.90	16	6	21	0.22	0.97	120.0	4020.00	2491.20	0.38	**5.00**	0.00	0.00	1.3
38	sm	21	1.00	40	120.0	4560.00	2750	0.84	1.00	1.00	21.00	0.86	18	6	24	0.26	0.96	120.0	4560.00	2750.40	0.38	**0.69**	1.29	0.54	1.3
41.5	sp	40	1.00	5	120.0	4980.00	2952	0.82	1.00	1.00	40.00	0.88	35	0	35	1.16	0.90	120.0	4980.00	2952.00	0.41	**2.84**	0.00	0.00	0.8
44.5	sm	21	1.00	20	120.0	5340.00	3125	0.80	1.00	1.00	21.00	0.81	17	4	22	0.23	0.95	120.0	5340.00	3124.80	0.38	**0.59**	1.43	0.77	0.8
49	sp	39	1.00	5	120.0	5880.00	3384	0.78	1.00	1.00	39.00	0.83	33	0	33	0.70	0.88	120.0	5880.00	3384.00	0.41	**1.72**	0.00	0.00	0.0
56	sm	50	1.00	35	120.0	6720.00	3787	0.74	1.00	1.00	50.00	0.83	42	6	47	94.39	0.81	120.0	6720.00	3787.20	0.43	**5.00**	0.00	0.00	0.0
58.5	sp	69	1.00	5	120.0	7020.00	3931	0.73	1.00	1.00	69.00	0.84	58	0	58	797217	0.80	120.0	7020.00	3931.20	0.43	**5.00**	0.00	0.00	0.0
63.5	cl	48	0.65	84.3	120.0	7620.00	4219	0.70	1.00	1.00	31.20	0.73	23	6	28	0.40	0.89	120.0	7620.00	4219.20	0.38	**5.00**	0.00	0.00	0.0
68.5	sp	40	1.00	5	120.0	8220.00	4507	0.68	1.00	1.00	40.00	0.74	30	0	30	0.47	0.84	120.0	8220.00	4507.20	0.39	**1.20**	0.00	0.00	0.0
71.5	sm	32	1.00	40	120.0	8580.00	4680	0.67	1.00	1.00	32.00	0.70	22	6	28	0.38	0.88	120.0	8580.00	4680.00	0.37	**1.04**	0.00	0.00	0.0
75	ml	32	1.00	66.8	120.0	9000.00	4882	0.65	1.00	1.00	32.00	0.68	22	6	27	0.36	0.87	120.0	9000.00	4881.60	0.36	**1.00**	0.00	0.00	0.0
76.5									1.00																

Settlement of Dry Sands Calculation Spreadsheet M.B. Dogan Last Updated: 6/14/2011 14:54

Settlement of Dry Sand Analysis

Project: Seal Beach

Project Number: NB11161340

Borehole and Drilling Data

Boring Number: BH-1 Driller Method: Mud Rotary Hammer Type: Auto
Elevation: Driller Name: Gregg Borehole Dia.: 4.5"
Depth to Groundwater: 9 Date Drilled: 5/4/2011 Total Depth: 76.5'

N =Blows Per foot
Ce =1.0 for Automatic

Cb =1.0 for 4.5 in. diam. Borehole

Lithology and Blowcount Data

Layer	Top (ft bgs)	Bottom (ft bgs)	Material (USCS)	Unit Weight	% Fines (Assumed)	N	Ce	Cb	Cr	Cl	Cs	N_{60}
1	0	2.5	ML	120	60	7	1	1	0.75	1.0	1.0	5
2	2.5	4	SC	120	30	21	1	1	0.85	1.0	1.0	17
3	4	5.5	SP	120	5	21	1	1	0.95	1.0	1.0	19
4	5.5	9	SM	120	15	31	1	1	0.95	1.0	1.0	29

Cr =0.75 rod 0 to 13 feet
=0.85 rod 13 to 20 feet
=0.95 rod 20 to 30 feet
=1.0 rod 30 to 100 feet
Cl =1.0 no liners

Cs =1.0 for SPT
=0.0.65 for CA rings

Table 4

Eathquake magnitude	Number of representative cycles at 0.65 Tmax	Volumetric strain ratio, $\epsilon_{C,N}/\epsilon_{C,N=15}$
8.5	26	1.25
7.5	15	1
6.75	10	0.85
6	5	0.6
5.25	2-3	0.4

M = 7.03
r_m = 0.92 (Table 4 - Tokimatsu & Seed 1987)
a_{max} = 0.45 g

σ'm=0.65*σ'o — From Fig 11 — From Fig 12 or 13 — multidirectional effect

Layer	Top Layer [ft]	Bottom Layer [ft]	Depth to m.p. [ft]	N_{60} blows	C_N	$(N_{60})_1$ blows	Dr (%)	fines correction kspt	(N60)1 (fines)	unit weight [pcf]	$(K_2)_{max}$	Gmax [psf]	overburden [psf]	eff. overburden [psf]	σ'm [psf]	σ'm [tsf]	γeff(Geff/Gmax)	γeff	γeff (%)	ЄC,M=7.5 (%)	ЄC,M (%)	2ЄC,M (%)	r_d	Settlement [in]
1	0	2.5	1.25	5	2.00	10	73	0.5	20	120	43	425	150.0	150.0	97.50	0.05	6.82E-05	1.2E-04	0.01	0.030	0.028	0.055	1.00	0.02
2	2.5	4	3.25	17	2.00	34	125	0.4	58	120	65	1032	390.0	390.0	253.50	0.13	7.27E-05	1.1E-04	0.01	0.005	0.005	0.009	0.99	0.00
3	4	5.5	4.75	19	1.87	36	122	0.0	36	120	66	1266	570.0	570.0	370.50	0.19	8.81E-05	1.2E-04	0.01	0.005	0.005	0.009	0.98	0.00
4	5.5	9	7.25	29	1.52	44	127	0.2	53	120	71	1679	870.0	870.0	565.50	0.28	9.83E-05	1.4E-04	0.01	0.003	0.003	0.006	0.98	0.00

sum= **0.02** [in]

$$D_r = 100\left(\frac{N_{SPT}}{12\sigma'_{vo}+17}\right)^{0.5}$$

Gibbs and Holts 1957

Settlement of Dry Sands Calculation Spreadsheet M.B. Dogan Last Updated, 6/14/2011 14:54

Settlement of Dry Sand Analysis

Project: Seal Beach

Project Number: NB11161340

Borehole and Drilling Data

Boring Number:	BH-2	Driller Method:	Mud Rotary	Hammer Type:	Auto
Elevation:		Driller Name:	Gregg	Borehole Dia.:	4.5"
Depth to Groundwater:	9	Date Drilled:	5/5/2011	Total Depth:	78.5'

N =Blows Per foot
Ce =1.0 for Automatic

Cb =1.0 for 4.5 in. diam. Borehole

Lithology and Blowcount Data

Layer	Top (ft bgs)	Bottom (ft bgs)	Material (USCS)	Unit Weight	% Fines (Assumed)	N	Ce	Cb	Cr	Cl	Cs	N_{60}
1	0	7.5	SM	120	35	5	1	1	0.75	1.0	1.0	3

Cr =0.75 rod 0 to 13 feet
=0.85 rod 13 to 20 feet
=0.95 rod 20 to 30 feet
=1.0 rod 30 to 100 feet
Cl =1.0 no liners

Cs =1.0 for SPT
=0.65 for CA rings

Table 4

Eathquake magnitude	Number of representative cycles at 0.65 Tmax	Volumetric strain ratio, $\epsilon_{C,N}/\epsilon_{C,N=15}$
8.5	26	1.25
7.5	15	1
6.75	10	0.85
6	5	0.6
5.25	2-3	0.4

M = 7.03
r_m = 0.92 (Table 4 - Tokimatsu & Seed 1987)
a_{max} = 0.46 g

σ'm=0.65*σ'o — From Fig 11 — From Fig 12 or 13 — multidirectional effect

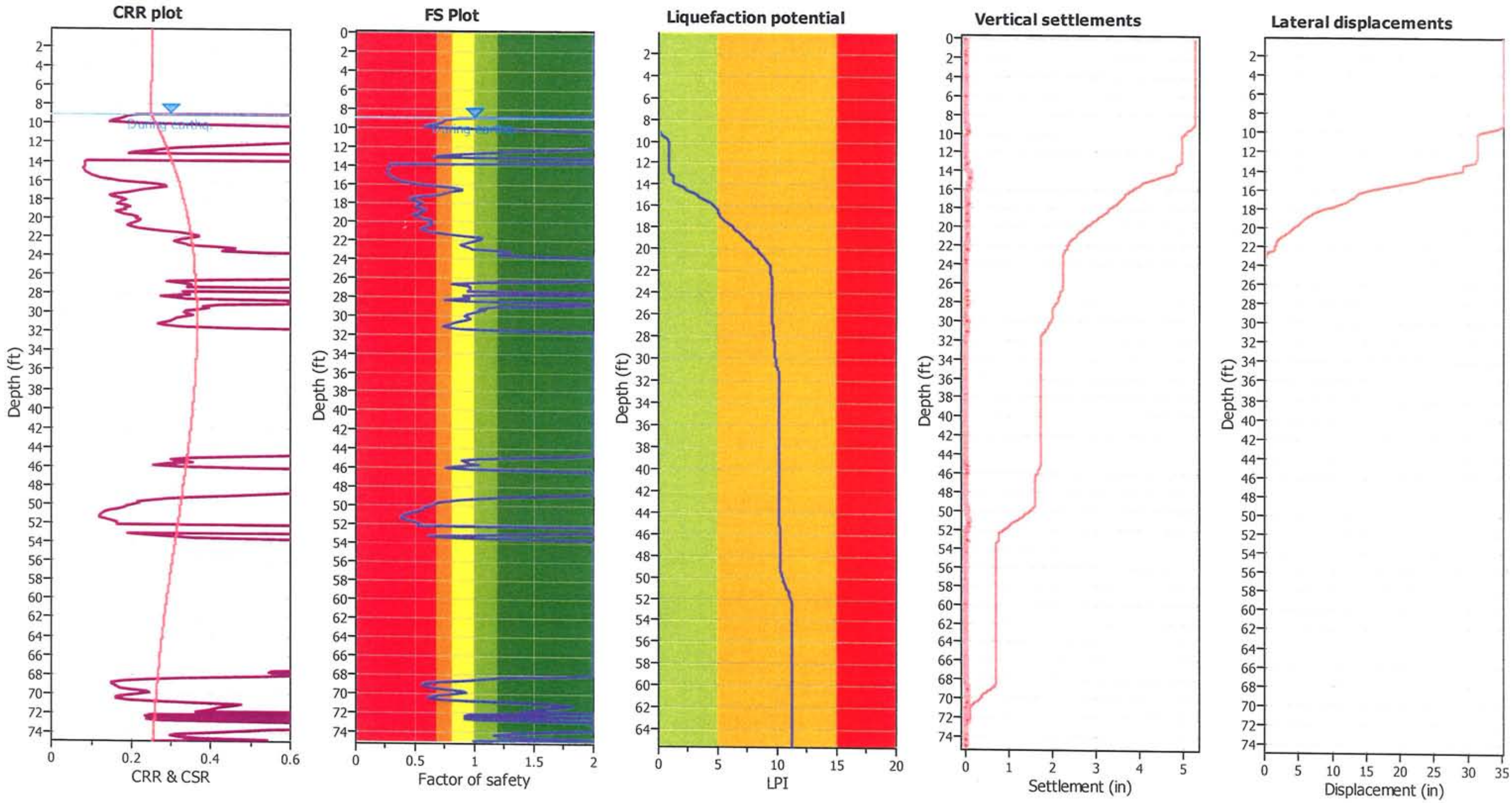
Layer	Top Layer [ft]	Bottom Layer [ft]	Depth to m.p. [ft]	N_{60} blows	C_N	$(N_{60})_1$ blows	Dr (%)	fines correction kspt	(N60)1 (fines)	unit weight [pcf]	$(K_2)_{max}$	Gmax [psf]	overburden [psf]	eff. overburden [psf]	σ'm [psf]	σ'm [tsf]	γeff(Geff/Gmax)	γeff	γeff (%)	ЄC,M=7.5 (%)	ЄC,M (%)	2ЄC,M (%)	r_d	Settlement [in]
1	0	7.5	3.75	3	2.00	6	52	0.5	12	120	36	622	450.0	450.0	292.50	0.15	1.39E-04	5.0E-03	0.50	1.000	0.920	1.640	0.99	1.66

sum= 1.66 [in]

$$D_r = 100\left(\frac{N_{SPT}}{12\sigma'_{vo}+17}\right)^{0.5}$$

Gibbs and Holts 1957

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER 1998
Fines correction method:	Robertson & Wride
Points to test:	Based on Ic value
Earthquake magnitude M_w:	7.03
Peak ground acceleration:	0.46
Depth to water table (insitu):	9.00 ft
Depth to water table (erthq.):	9.00 ft
Average results interval:	3
Ic cut-off value:	2.60
Unit weight calculation:	Based on SBT
Use fill:	No
Fill height:	N/A
Fill weight:	N/A
Transition detect. applied:	No
K_σ applied:	No
Clay like behavior applied:	Sands only
Limit depth applied:	No
Limit depth:	N/A

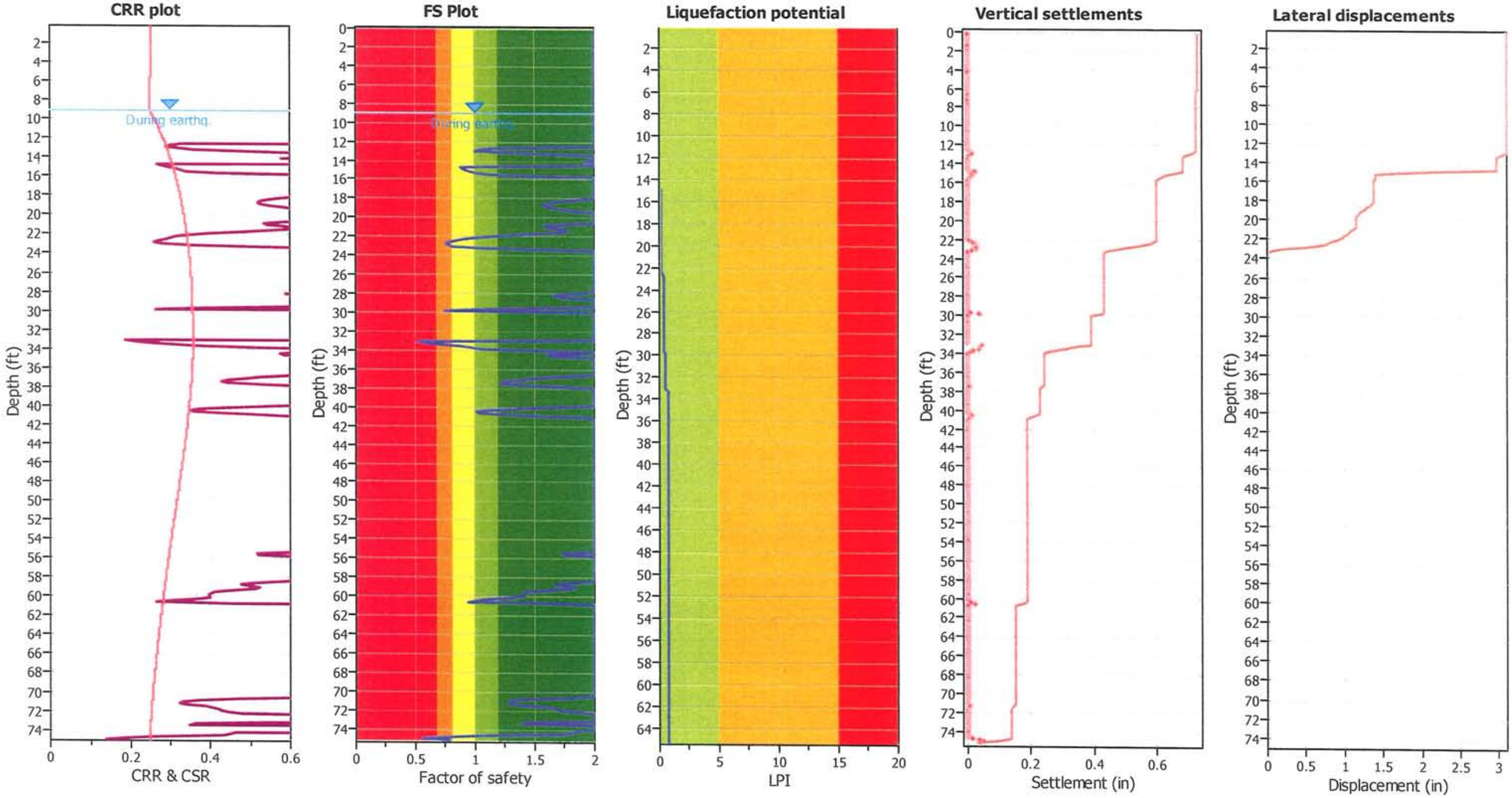
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER 1998	Depth to water table (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	Robertson & Wride	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.03	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.46	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
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LPI color scheme

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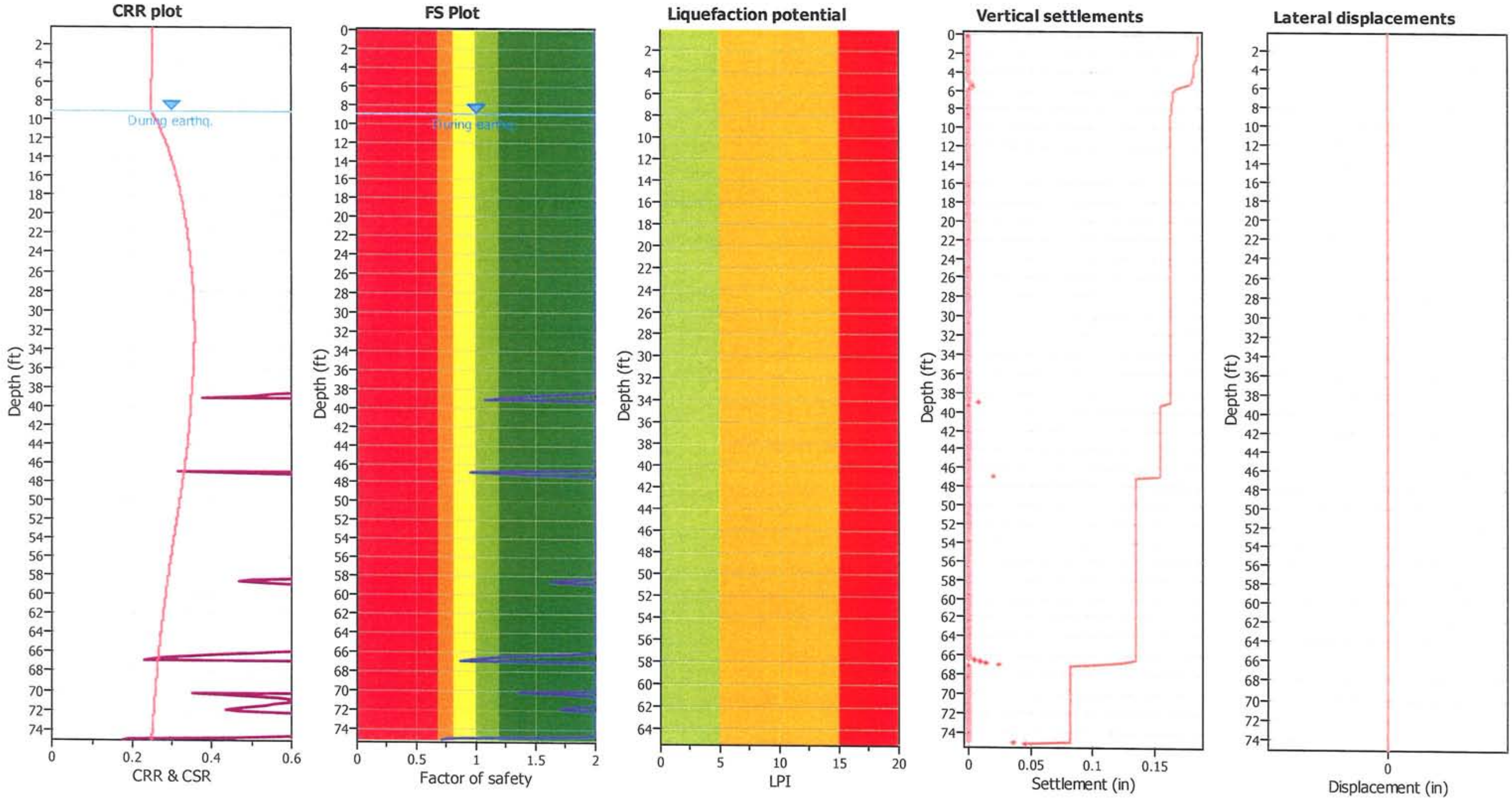
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER 1998	Depth to water table (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	Robertson & Wride	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.03	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.46	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
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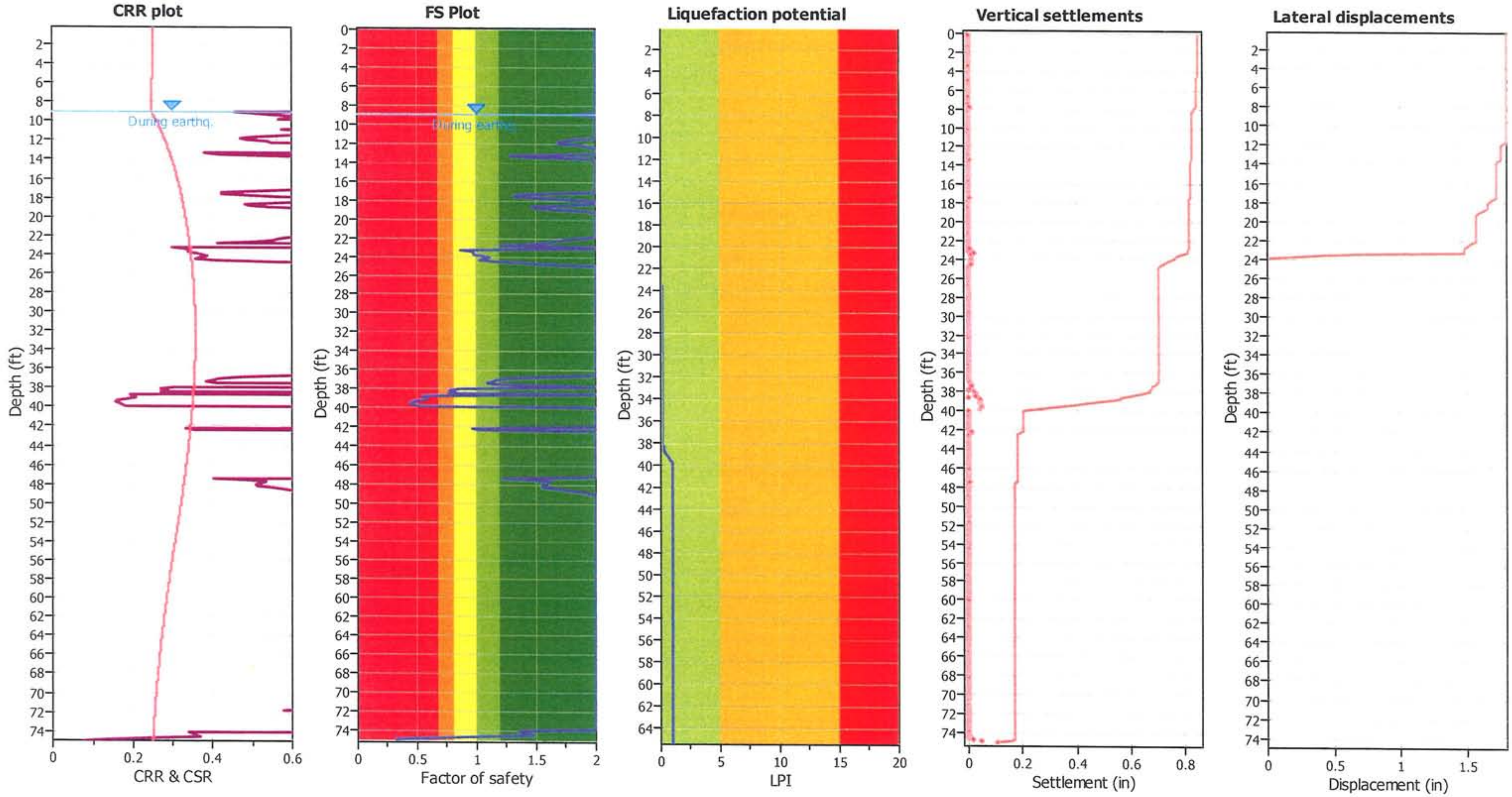
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER 1998	Depth to water table (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	Robertson & Wride	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.03	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.46	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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LPI color scheme

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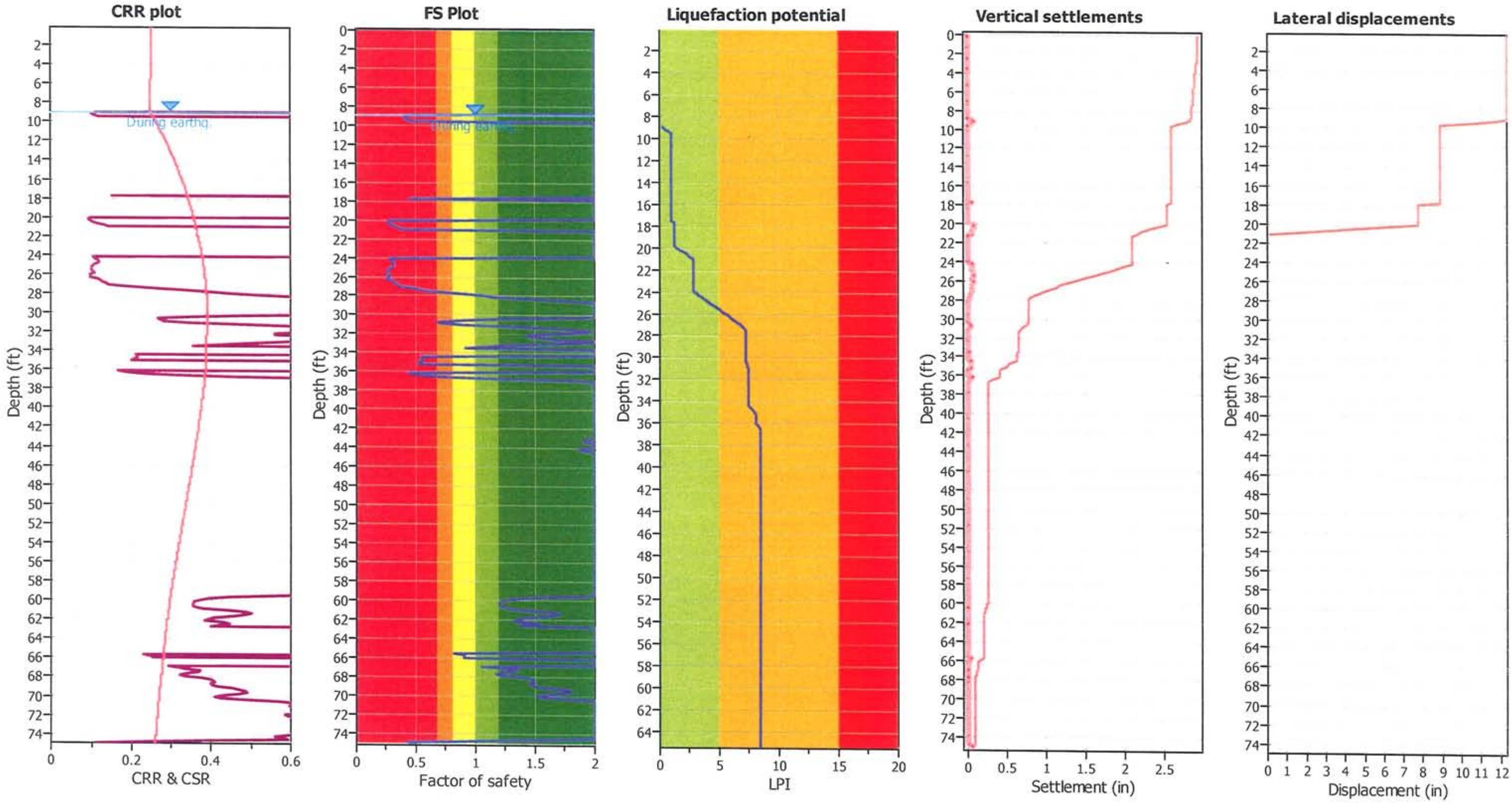
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER 1998	Depth to water table (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	Robertson & Wride	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.03	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.46	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

APPENDIX D

Seismic Slope Stability Analysis

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geoenvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.09		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.08	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.12	seconds	
Moment Magnitude (Mw)	7.0		
Spectral Acceleration (Sa(1.5Ts))	1.15	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	5	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	62.64	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	0.000		eq. (3)
D1	32.50	cm	calc. using eq. (7)
D2	62.64	cm	calc. using eq. (7)
D3	120.76	cm	calc. using eq. (7)
P(D>d_threshold)	1.000		eq. (7)

Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geoenvironmental Engineering, Vol 133, No. 4, pp. 381-392, April 2007

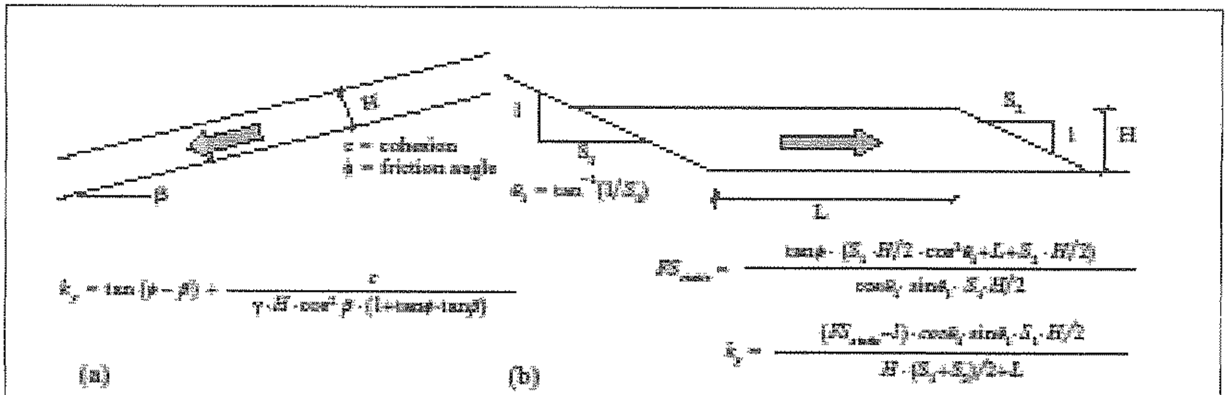


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

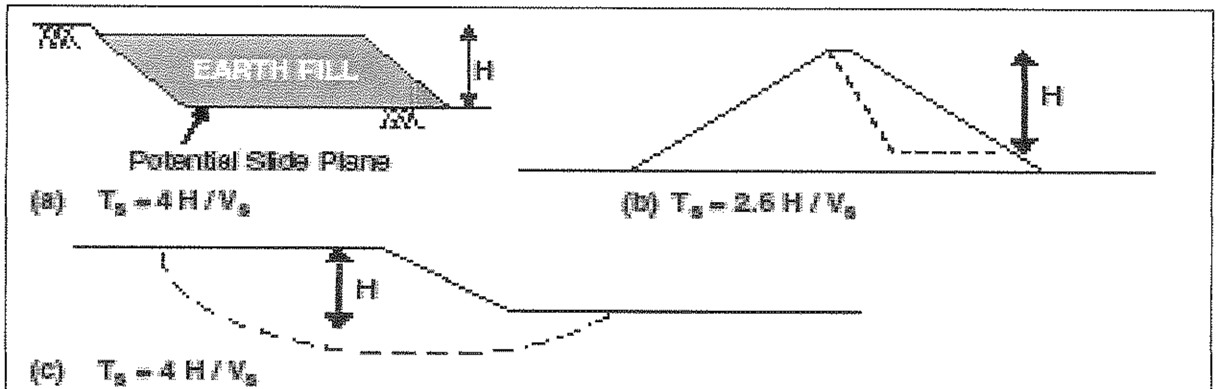
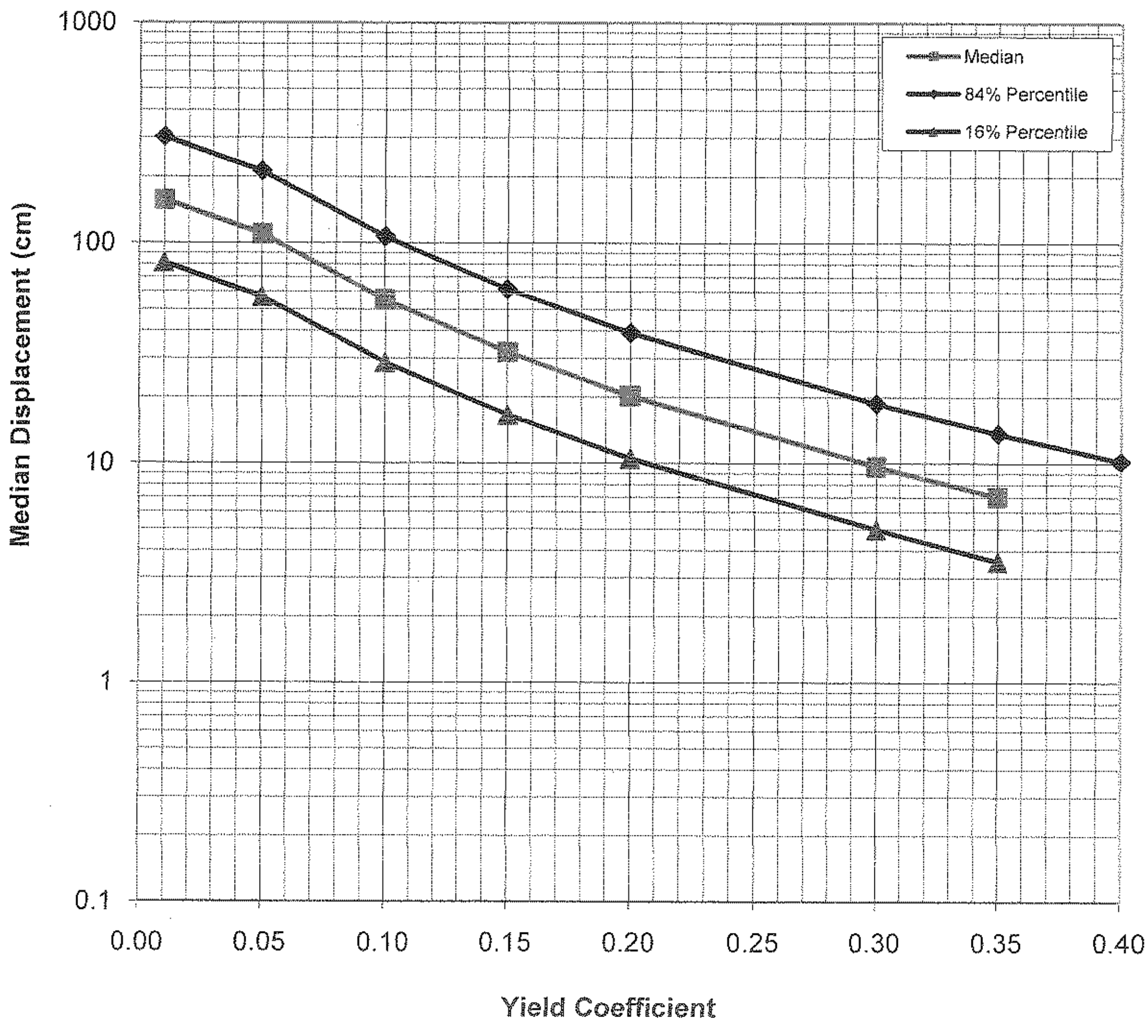


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray, J.D. (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, Vol. 6, Pitilakis, Kyriazis D., Ed., Springer, Vol. 6, pp. 327-353.

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.010	0.00	156.6	156.6	301.9	81.2
0.05	0.00	109.7	109.7	211.4	56.9
0.1	0.00	55.3	55.3	106.6	28.7
0.15	0.00	31.9	31.9	61.6	16.6
0.2	0.00	20.2	20.2	39.0	10.5
0.3	0.00	9.7	9.7	18.7	5.0
0.35	0.02	7.1	7.0	13.6	3.6
0.4	0.04	5.4	5.2	10.2	2.5



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Seal Beach DWP Site Specific Plan EIR

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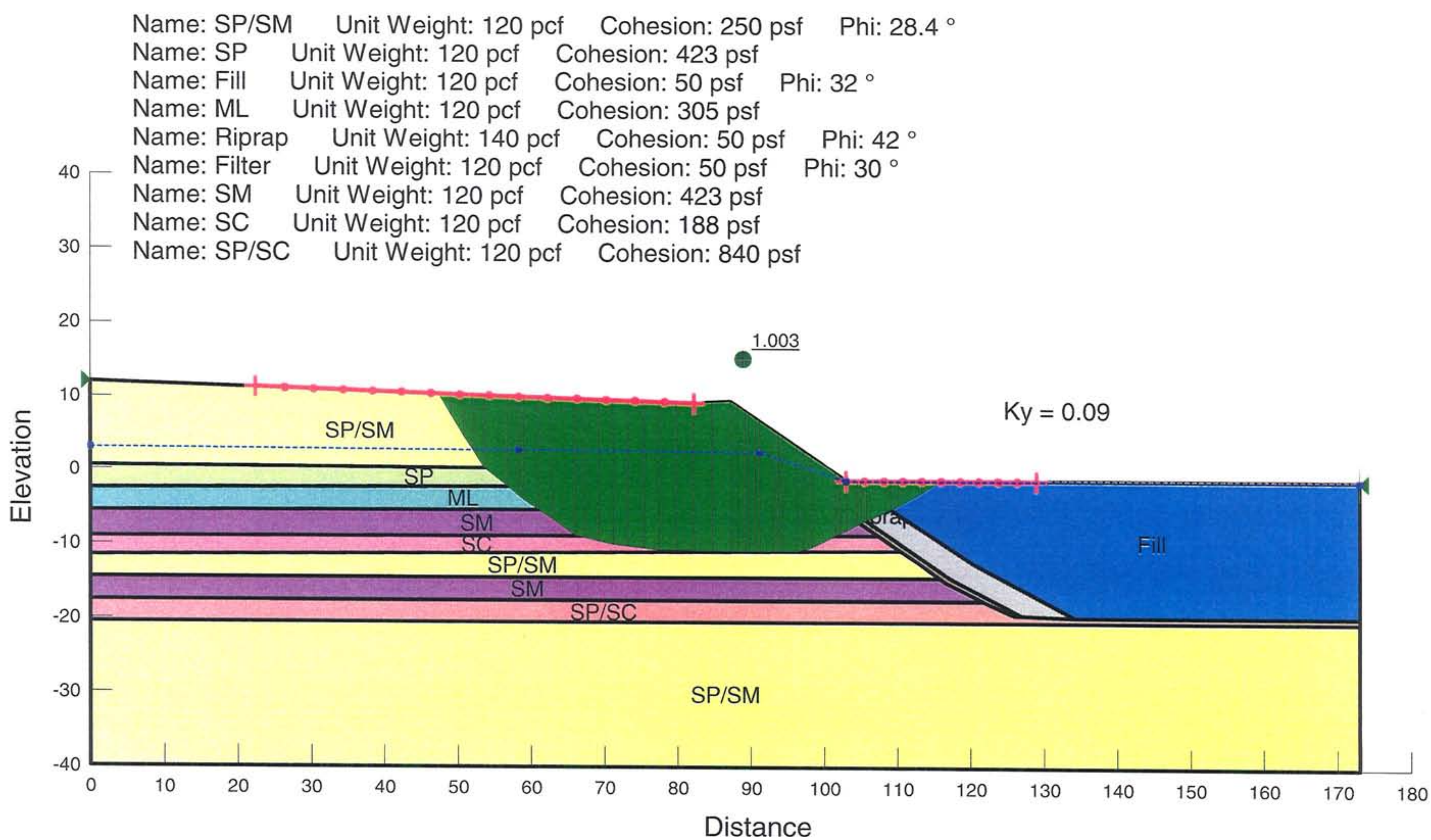

PRELIMINARY GEOTECHNICAL EVALUATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT
SEAL BEACH, ORANGE COUNTY, CALIFORNIA

PREPARED FOR

CELEBRATE HOMES
1160 HACIENDA DRIVE
VISTA, CALIFORNIA 92083

PREPARED BY

GEOTEK, INC.
1384 POINSETTIA AVENUE
VISTA, CALIFORNIA 92081

PROJECT NO.: 2888SD3 SEPTEMBER 12, 2005

GEO TEK, INC.
1384 Poinsetta Ave., Suite A
Vista, CA 92081-8505
(760) 599-0509 FAX (760) 599-0593

Geotechnical
Environmental
Materials

September 12, 2005
Project No.: 2888SD3

Celebrate Homes
1160 Hacienda Drive
Vista, California 92083

Attention: Mr. Rick Snyder

Subject: Preliminary Geotechnical Evaluation
Proposed Residential Development
Ocean Avenue and Marina Drive
Seal Beach, Orange County, California

Dear Mr. Snyder:

We are pleased to provide herewith the results of our preliminary geotechnical evaluation for the proposed residential development located west of the intersection of First Street and Marina Drive, in the City of Seal Beach, Orange County, California. This report presents the results of our exploration, discussion of our findings, and provides preliminary geotechnical recommendations for foundation design and construction. In our opinion, site development appears feasible from a geotechnical viewpoint provided that the recommendations included herein are incorporated into the design and construction phases of development.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call our office.

Respectfully submitted,
GeoTek, Inc.

REGISTERED PROFESSIONAL ENGINEER
WILLIAM R. MORRISON
No. 2468
Exp. 12/31/06
GEOTECHNICAL
STATE OF CALIFORNIA

Jeffrey P. Blake,
CEG 2248, Exp. 10/31/05
Geotechnical Department Manager

William R. Morrison
William R. Morrison,
RGE 2468, Exp. 12/31/06
Senior Geotechnical Engineer

(5) Addressee
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TABLE OF CONTENT

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1. INTENT

It is the intent of this report to aid in the design and completion of the proposed development. Implementation of the advice presented in Section 6 of this report is intended to reduce risk associated with construction projects. The professional opinions and geotechnical advice contained in this report are not intended to imply total performance of the project or guarantee that unusual or variable conditions will not be discovered during or after construction.

The scope of our evaluation is limited to the explored area that is shown on the Boring Location Plan (Figure 2). This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to us by the client. Further, no evaluation of any existing site improvements is included. The scope is based on our understanding of the project and the client's needs, and geotechnical engineering standards normally used on similar projects in this region.

2. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the overall geotechnical conditions on the site. Services provided for this study included the following:

- Research and review of available geologic data and general information pertinent to the site,
- Site exploration consisting of the excavation, logging, and sampling of 5 exploratory borings,
- Laboratory testing on representative samples collected during the subsurface exploration,
- Review and evaluation of site seismicity,
- Geotechnical evaluation of the field data and laboratory data, and
- Compilation of this geotechnical report which presents our findings, conclusions, and recommendations for site development.

It should be understood that our proposed scope of services discussed herein is intended to provide a preliminary evaluation of the Geotechnical conditions at the site to assist in Celebrate Homes' assessment of the site's feasibility for the proposed development. An additional design level Geotechnical Evaluation of the site appears to be warranted to provide specific recommendations for the proposed development once more detailed plans for the proposed development become available.

3. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

3.1 SITE DESCRIPTION

The subject site is located in the City of Seal Beach in the County of Orange, California. More specifically, the site is located west of the intersection of First Street and Marina Drive (see Figure 1). The property is currently vacant land and encompasses approximately 10-acres. The site is bounded to the north by the San Gabriel River outlet, to the west by an asphalt parking area for Seal Beach, to the south by First Street and Ocean Avenue, and to the east by Marina Drive. The site topography is generally flat to gently sloping toward the north. Based on our review of a topographic survey plan prepared by Coast Surveying, Inc., the elevation of the site ranges from approximately 10 feet to 18 feet above mean sea level. Further information regarding site location and existing topography is shown on Figure 2.

3.2 PROPOSED DEVELOPMENT

We understand that the proposed development at the site will consist of the construction of residential structures and associated improvements. No preliminary grading plans were provided at the time of this report, however grading consisting of cuts and fills of less than 10 feet are anticipated for the majority of the site. We anticipate that the proposed structures will be of wood framed construction. The foundations for the proposed structures are anticipated to consist of both conventional slab-on-grade construction and special foundation systems (such as mat foundations, post-tensioned slabs, and/or drilled pier foundation systems). Structural loads for the proposed residences are not known at this time. However, we have assumed that the structural loads will be typical for wood frame buildings. The recommendations presented herein should be reviewed once additional information regarding the anticipated structural loads is available.

4. FIELD EXPLORATION AND LABORATORY TESTING

4.1 FIELD EXPLORATION

A field exploration, consisting of the drilling, logging, and sampling of 5 exploratory borings, was conducted on August 22 and 23, 2005. The exploratory borings were excavated with a truck-mounted drill rig (CME 75) to a maximum depth of 59 feet below existing grade. A geologist from our firm logged the excavations and collected samples for use in laboratory testing. The logs of our exploratory borings are included in Appendix A. The locations of exploratory borings are shown on the Boring Location Plan (Figure 2).

4.2 LABORATORY TESTING

Laboratory testing was performed on selected disturbed and relatively undisturbed samples collected during the field exploration. The purpose of the laboratory testing was to confirm the field classification of the soil materials encountered and to evaluate their physical properties for use in the engineering design and analysis. The results of the laboratory-testing program along with a brief description and relevant information regarding testing procedures are included in Appendix B.

5. GEOLOGIC AND SOILS CONDITIONS

5.1 GENERAL

A brief description of the earth materials encountered is presented in the following sections. A more detailed description of these materials is provided on the exploratory boring logs included in Appendix A. Based on our site reconnaissance, subsurface excavations, and review of published geologic maps, the site is underlain to the maximum depth explored by what is considered to be Quaternary-aged alluvial deposits. An artificial fill layer of variable thickness overlies the alluvium.

5.1.1 Artificial Fill

The encountered portions of the existing fill materials were noted to range in thickness from between 3 feet to approximately 35 feet. The thickest portions of fill materials appear to be dredged materials associated with the previous construction of the San Gabriel River channel. As encountered in our exploratory borings, the fill materials typically consist of gray, poorly graded sands with variable amounts of silt and clay, along with clayey to sandy silts. The sandy portions of the fill were observed to be dry to saturated and loose to medium dense, while the more cohesive fine-grained portions were noted to be damp to saturated and soft to stiff, at the time of our subsurface exploration.

5.1.2 Alluvium

Quaternary-aged alluvial deposits underlie the existing fill materials and were observed to extend to the maximum explored depth of 59 feet below existing grade. In general, the alluvium was observed to consist of yellow-brown to gray-brown sand interbedded with clayey silt. These materials were noted to exhibit iron oxide staining and were generally observed to be moist to saturated and medium dense to dense within the sandy portions and moist to saturated and stiff to hard within the more cohesive fine-grained portions at the time of our subsurface exploration.

5.2 SURFACE AND GROUND WATER

5.2.1 Surface Water

If encountered, surface water on this site is the result of direct precipitation or surface run-off from surrounding sites. Site drainage is predominately by sheet flow in a westerly direction. All site drainage should be reviewed and designed by the project civil engineer.

5.2.2 Groundwater

Groundwater was encountered in all of our exploratory borings at depths of between 9 and 15 feet below existing grades. Fluctuations in the groundwater level should be anticipated mainly due to variations in rainfall, temperature, and other factors not evident at the time of our subsurface exploration. In general, groundwater can be anticipated to be a constraint to site development during remedial grading activities, building foundation construction, and utility trench line construction.

5.3 FAULTING AND SEISMICITY

The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor was evidence of active faulting observed during our subsurface exploration at the site, The site situated within an Alquist-Priolo Earthquake Fault Zone (Special Studies Zone).

A computer program EQFAULT (Blake, 2000a) was used to approximate the distance to known late Quaternary faults and estimate peak ground accelerations (PGA) at this site based on a deterministic analysis. The program applies a user-selected attenuation relationship to calculate the PGA's that may result from the maximum earthquakes on each of the faults found in the search radius. The Compton Thrust Fault, located within 0.25 miles of the site, and the L.A. Basin segment of the Newport-Inglewood Fault, located approximately 1.1 mile east of the site, are considered to represent the highest risk to generate ground shaking. A maximum earthquake magnitude of 6.9 along the L.A. Basin segment of the Newport-Inglewood Fault and an estimated peak site acceleration of 0.68g are postulated.

The computer program FRISKSP (Blake, 2000) was also used to estimate peak ground acceleration (PGA) based on a probabilistic analysis. The PGA values, which correspond to a 10 percent probability of exceedance in 50 years, are on the order of 0.58g, based on attenuation relations by Boore Et Al. (NEHRP D, 1997).

5.4 LIQUEFACTION EVALUATION

Liquefaction occurs when saturated sands and silts lose their physical strengths when subjected to ground shaking. Liquefaction potential is primarily affected by material gradation, relative density, and intensity and duration of ground motion.

We have evaluated the potential for liquefaction at the site in accordance with the procedure recommended by The National Center For Earthquake Engineering Research (Youd, et al, 2001). Our evaluation incorporates the geotechnical data obtained from our exploratory borings and utilizes the earthquake induced ground motion having a 10 percent probability of exceedance in 50 years (475-year return period).

As noted above, our probabilistic seismic ground motion of the site analysis yielded a peak ground acceleration of approximately 0.58g for an event having a 475-year return period. When scaling this ground acceleration with respect to a magnitude 7.5 earthquake (the magnitude to which liquefaction susceptibility analyses are applicable), the FRISKSP software (Blake, 2000) yields a peak ground acceleration of 0.47g, which is the value utilized in our evaluation.

As discussed previously, ground water was encountered in all of our exploratory borings at depths on the order of between 10 and 15 feet below existing site grades. For analysis purposes, our liquefaction evaluation incorporates a ground water depth of 9 feet below existing grade.

The results of our calculations (Appendix C) indicate that during a design level earthquake along a nearby fault, the sand and silt layers encountered in borings B-1 and B-4 at various depths between the existing surface and 56 feet below the existing ground surface are susceptible to liquefaction.

Seismically induced settlement can occur due to reorientation of soil particles during strong shaking of unsaturated sands, as well as in response to liquefaction of saturated loose granular soils. The potential seismically induced settlement within the upper alluvial soils was estimated using the Tokimatsu and Seed procedure (1987). Our evaluation was based on the ground motion generated by a seismic event having a 10 percent probability of exceedance in 50 years (corrected peak ground acceleration of 0.47g). Based on our evaluation, we estimate the total seismic-induced settlement to be on the order of 4 - 8 inches.

Lateral spreading often results from the liquefaction of loose granular soils at depth. Due to the observed presence of a significant channel (San Gabriel River outlet) in the immediate vicinity of the site, liquefaction-induced lateral spreading at the site appears to be a potential concern.

Based on the subsurface data disclosed by our exploratory borings B-1 and B-4, we have performed a preliminary estimate of liquefaction induced lateral spreading that could occur toward the San Gabriel River Channel. Using Youd, Hansen and Bartletts Procedure (2002), we estimate lateral spreading of as much as 2 to 4 feet could occur at a point located approximately 100 feet from the channel as a result of a design magnitude seismic event. This displacement can be expected to decrease as the distance from the channel increases.

5.5 OTHER SEISMIC HAZARDS

As discussed previously, the site is relatively level with no significant slopes on or directly adjacent to the site. Evidence of ancient landslides or slope instability at this site was not observed during our investigation. Thus, the potential for landslides is considered low at this site.

Secondary seismic hazards such as seiches and tsunamis are often associated with seismic events. Due to the site's relatively low elevation and proximity to an open body of water, the site could be affected by a tsunami. Houston and Garcia (1974) estimate that the area could be subject to 500-year run up heights on the order of 10 to 11 feet.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

The proposed development of the site appears feasible from a geotechnical viewpoint provided that the following recommendations are incorporated into the design and construction phases of development. The most significant geotechnical considerations that will warrant mitigation are related to the relatively shallow groundwater level, potential for earthquake-induced ground shaking, and the presence of potentially compressible fill materials. These considerations include the effects of liquefaction including dynamic settlement and lateral spreading of relatively loose sands and non-plastic silts within the alluvium that underlies the site, and incomplete removal and subsequent settlement of potentially compressible soils due to the presence of high groundwater. Consequently, we are recommending conventional shallow foundation systems where complete removal and recompaction of the upper, potentially compressible materials can be readily performed, and special foundation systems such as mat foundations, post-tensioned slabs, or drilled pier foundation systems for the support of the proposed residential structures where remedial earthwork is not feasible. Other methods to mitigate these potential geotechnical hazards, such as ground improvement, are also feasible. However, the implementation of an alternative mitigative measure can be expected to significantly increase construction costs. If desired, we would be pleased to provide recommendations for the design/implementation of ground improvement upon request.

As discussed above, the recommendations contained herein are based on a preliminary evaluation of the geotechnical conditions. Once more detailed plans for the proposed development become available, we recommend that an additional design-level geotechnical evaluation of the site be performed to provide more specific recommendations for the proposed development.

6.2 EARTHWORK CONSIDERATIONS

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the 2001 California Building Code (CBC), and our recommendations contained in this report. The Grading Guidelines included in Appendix D outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix D.

6.2.1 Site Clearing

In areas of planned grading or improvements, the site should be cleared of vegetation, roots and debris, and properly disposed of offsite. Any holes resulting from site clearing, tree removal, and/or the backhoe trenches excavated during this study should be replaced with properly compacted fill materials.

6.2.2 Fills

The onsite soil materials are considered suitable for reuse as compacted fill provided they are free from vegetation, debris, rocks larger than 6 inches in maximum dimension, and other deleterious material. Any import fill should consist of relatively low-expansive soils (EI<50) that are evaluated by our firm prior to arrival at the site. The fill materials should be compacted in layers no thicker than 8 inches to at least 90 percent of maximum dry density with a moisture content of at least optimum, as determined in accordance with ASTM Test Method D1557-00. Those areas to receive fill should be scarified to a depth of 8 inches; moisture conditioned to at least optimum moisture content and recompacted to at least 90 percent of maximum dry density.

6.2.3 Removals

Based on the results of our subsurface exploration, the existing fill materials appear to be relatively loose and potentially compressible. As such, they are considered unsuitable for the support of settlement-sensitive structures or additional fill in their current condition and should be subject to complete removal and recompaction within the limits of grading. In those areas where the depth of existing fill materials extends below the groundwater table, complete removals of the fill materials are not feasible. In these areas, we recommend that the upper 8 to 10 feet of soil, along with organic and other deleterious materials, be removed.

Upon removal of the upper soils within those areas exhibiting a shallow ground water surface, saturated and yielding subgrade conditions may be encountered. In order to help stabilize the yielding subgrade soils within the bottom of the removal areas, the contractor can consider the placement of uniform sized, ¾-inch crushed rock within the area exhibiting the "pumping" conditions. The crushed rock should be properly tracked into the underlying soils such that it is adequately intruded into and interlocks with the soils. We expect that between a 6- and 12-inch thickness of the crushed rock will be required; however, this should be further evaluated during construction. Following the placement and tracking of the gravel layer into the underlying "pumping" soils, it is recommended that Mirafi 600X stabilization fabric (or approved equivalent) then be placed upon the gravel layer. Fill soils should then be placed upon the fabric and compacted to a minimum 90 percent relative compaction (based on ASTM test method D1557) until finished grades are reached. The gravel and stabilization fabric should extend at least 5 feet laterally beyond the limits of the "pumping" areas. These operations should be performed under the observation and testing of a representative of Geotek, Inc. in order to evaluate the effectiveness of these measures and to provide additional recommendations for mitigative measures, as warranted.

Due to the incomplete removal of potentially compressible fill soils, some settlement could occur following the placement of fill. As a result, we recommend that following the completion of rough grading at the site, settlement monuments should be installed at finish rough grade. These monuments should be established based on a known bench mark and their elevations should be monitored by a licensed land surveyor on a weekly basis. The surveyor's settlement monument data should be reviewed weekly by the Geotechnical Engineer. This monitoring should continue until the consolidation is deemed to have sufficiently stabilized. Once it has been concluded that the remaining settlement is within acceptable levels, the settlement monuments may be destroyed and fine grading may proceed.

6.2.4 Excavation Characteristics

Refusal to our drilling equipment was encountered in our exploratory boring B-2, that was drilled near the western margin of the site at approximately 22 feet below existing grade. Based on the subsurface exploration performed, the presence of resistant material at depth appears to be localized and limited to the portion of the site adjacent to the San Gabriel River channel. The encountered refusal material could be hard rock boulders (rip-rap) associated with the original construction along the banks of the river channel. Although site grading plans were not available for review at the time of this report, construction depths are not anticipated to extend below the depths where these materials were encountered. As such, the on-site soils are anticipated to be rippable with conventional earthmoving equipment within the anticipated construction depths.

All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with OSHA guidelines.

6.2.5 Expansive Soils

Based on the laboratory test results, the materials encountered during our subsurface exploration are anticipated to possess a low to medium expansion potential (EI<91).

Placement of any clayey soils within three feet of finish grades should be avoided. Mixing of these soils with more granular materials that can be found onsite may reduce their potential for expansion.

6.3 FOUNDATION SUPPORT

6.3.1 Conventional Foundation Recommendations

In the areas where complete removal and recompaction of the upper soils can be accomplished, the proposed residential structures can be supported on conventional continuous or isolated spread footings bearing entirely upon properly compacted fill materials. Foundations supporting single story structures should be constructed with an embedment of at least 12 inches below finish grade, while those supporting two-story structures should be constructed with an embedment of at least 18 inches below finish grade. At these depths, footings may be designed for an allowable soil bearing value of 2,000 psf. This value may be increased by one-third for loads of short duration, such as wind and seismic forces. Continuous footings supporting single-story structures should have a minimum width of 12 inches, while those supporting two-story structures should have a minimum width of 15 inches. Based on geotechnical considerations, footings should be provided with reinforcement consisting of two No. 4 rebars, one top and one bottom. We recommend a minimum width of 24 inches for isolated spread footings.

Passive resistance to lateral loads can be computed as an equivalent fluid pressure having a density of 250 psf per foot of depth to a maximum earth pressure of 3000 psf. A coefficient of friction between soil and concrete of 0.30 may be used with dead load forces. When combining passive and frictional resistance, the passive pressure component should be reduced by one-third.

6.3.2 Special Foundation Systems

In the areas where incomplete removals are performed and/or the potential for seismically-induced differential settlement exists, special foundation systems such as mat foundations, post-tensioned slabs, or drilled pier foundation systems may be viable options for support of the proposed residential structures.

The potential effects of post-construction and seismic-induced settlement on the proposed structures can be reduced by the use of a structural mat foundation. Structural mat foundations can be expected to provide relatively uniform settlement across a structure. Mat foundations should be designed to bridge over voids that may develop under the slab due to differential settlement. The mat foundation should be founded within compacted fill materials, with a minimum embedment of 18 inches below finish grade. For mats founded on soft, wet or cohesionless soils, special preparation of the bottom will likely be required to support construction traffic.

A rigid reinforced concrete mat can be used to support the high structural loads on properly compacted fill. Mat foundations should be properly reinforced to form a relatively rigid structural unit in accordance with the structural engineers design. For preliminary design purposes, an uncorrected modulus of subgrade reaction of 100 pounds per cubic inch (pci) can be assumed. For large foundations, the modulus is typically reduced by 75 percent (i.e., to 25 pci). Actual geotechnical design parameters can be provided upon completion of a more complete geotechnical evaluation of the proposed building site.

Another foundation system that, if properly designed and constructed, can resist the distress related to post-construction and/or seismic induced differential settlements is a post-tensioned slab. The structural design of post-tensioned slabs should follow the recommendations of the Post-Tensioning Institute (PTI) Method and Section 1819 of the 2001 California Building Code (CBC).

Based on the geotechnical data acquired during our subsurface exploration, we recommend that an allowable bearing capacity of 1,500 pounds per square feet (psf), and a slab-subgrade friction coefficient of 0.75 be used for design of post-tensioned slabs. Final design should be verified based upon actual soil conditions encountered and results of laboratory testing performed during or at the completion of site grading.

Drilled piers bearing within relatively competent alluvial materials may also be utilized for the support of the proposed building loads in areas where complete removal and recompaction of the existing fill materials cannot be achieved. The drilled piers can be designed utilizing either end-bearing or skin friction design. Drilled piers should be embedded at least 5 feet within the alluvial materials or 14 feet below the existing ground surface (whichever is deeper). At these depths, preliminary estimates for allowable end-bearing piers are on the order of 3,000 to 4,000 psf at the tip of the drilled pier. The preliminary allowable bearing capacity incorporates a factor-of-safety of at least 3 and can usually be increased by one-third for loads of short duration, including wind and seismic forces.

We estimate that drilled piers that derive their support by skin friction and are also founded at least 5 feet within the alluvial materials or 14 feet below the existing ground surface (whichever is deeper) can typically be designed for an allowable skin friction on the order of 120 psf along the portion of the shaft of the pier bearing within relatively competent alluvial materials. An increase in allowable skin friction of approximately 4 psf can be anticipated for each additional foot of embedment within the alluvial materials.

Design of drilled piers subjected to earthquake loading should consider the effects of downdrag, due to the potential for liquefaction within portions of the fill. Downdrag loads are estimated to be on the order of 150 psf along that portion of the shaft above the water table and 500 psf along that portion of the shaft that extends through the liquefiable soils.

Because of the relatively high ground water level, along with the presence of poorly graded sands within the fill and alluvium, it will be necessary to utilize temporary casing or bentonite slurry to support the walls of the shaft prior to the placement of concrete. Further, the cleaning of loose slough from the bottom of the shaft excavation will be warranted for drilled piers that will derive their support from end-bearing conditions.

6.3.3 Settlement

As discussed above, we anticipate moderate settlement of the structure foundations under static loading provided the upper soils at the sites are removed and recompacted as described herein. Recommendations for the monitoring of settlement following rough grading are presented in Section 6.2.3 of this report We estimate that settlements due to foundation loading by the proposed residential structures under static conditions to be on the order of 1 to 2 inches total and ½ to 1 inch differential (across a 40 foot span).

As discussed in Section 5.4 of this report, we estimate that liquefaction of silt and sand layers within the fill due to a design earthquake could induce a total seismic settlement of approximately 4 to 8 inches. Utilizing the recommended foundation system for the proposed structures at the site, we estimate a seismic-induced differential settlement on the order of 3 to 4 inches across a 40-foot span.

6.3.4 Seismic Design Parameters

Seismically resistant structural design in accordance with local building ordinances should be followed during the design of all structures. Building Codes have been developed to reduce structural damage. However, some level of damage as the result of ground shaking generated by nearby earthquakes is considered likely in this general area.

For the purpose of seismic design a Type B seismic source (L.A. Basin segment of the Newport-Inglewood Fault) located less than 2 km from the site may be used. Table 6.3.2 below presents seismic design factors in keeping with the criteria presented in the 2001 CBC, Division IV & V, Chapter 16.

TABLE 6.3.2 – SEISMIC DESIGN PARAMETERS

Parameters	**Soil Profile Type**	**C_a**	**C_v**	**N_a**	**N_v**	**Seismic Source Type**
Source Table	16J	16Q	16R	16S	16T	16U
Value	S_D	0.57	1.02	1.3	1.6	B

6.3.5 Foundation Set Backs

Where applicable, the following setbacks should apply to all foundations. Any improvements not conforming to these setbacks may be subject to lateral movements and/or differential settlements:

- The outside bottom edge of all footings should be set back a minimum of H/3 (where H is the slope height) from the face of any descending slope. The setback should be at least 7 feet and need not exceed 20 feet.
- The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 projection upward from the bottom inside edge of the wall stem.
- The bottom of any existing foundations for structures should be deepened so as to extend below a 1:1 projection upward from the bottom of the nearest excavation.

6.3.6 Slab-On-Grade

Where applicable, concrete slabs (including the mat foundations recommended above) should be a minimum of 4 inches thick and reinforced as per structural engineer requirements. Control joints should be provided to help reduce random cracking. Slabs should be underlain by a 4-inch thick capillary break layer consisting of clean sand (S.E. 30 or greater). Where moisture condensation is undesirable, all slabs should be underlain with a minimum 6 mil polyvinyl chloride membrane, sandwiched between two layers of clean sand (S.E. 30 or greater), each being at least two inches thick. Care should be taken to adequately seal all seams and not puncture or tear the membrane. The sand should be proof rolled.

It should be noted that the above recommendation is based on soil support characteristics only. The structural engineer should design the actual slab and beam reinforcement based on expansion indices of the finish grade soils, actual loading conditions, and possible concrete shrinkage.

6.3.7 Soil Corrosivity

The soil resistivity at this site was tested in the laboratory on a representative sample collected during the field investigation. The results of the testing (Appendix B) indicate that the upper soils are highly corrosive to buried metallic structures. It is recommended that a corrosion engineer be consulted to provide recommendations for proper protection of buried metal pipes at this site.

6.3.8 Utilities

As discussed above, the project site appears to be susceptible to liquefaction and a considerable amount of seismically-induced settlement and lateral spreading. Consequently, consideration should be given to "flexible" design for on-site utility lines and connections.

Except where extending perpendicular to/under proposed foundations, utility trenches should be constructed outside a 1:1 projection from the base-of-foundations. Trench excavations for utility lines which extend under structural areas should be properly backfilled and compacted.

Utilities should be bedded and backfilled with clean sand or approved granular soil to a depth of at least 1-foot over the pipe. This backfill should be uniformly watered and compacted to a firm condition for pipe support. The remainder of the backfill shall be typical on-site soil or imported soil which should be placed in lifts not exceeding 8 inches in thickness, watered or aerated to 0 to 3 percent above the optimum moisture content, and mechanically compacted to at least 90 percent of maximum dry density (based on ASTM D1557).

6.4 CONCRETE CONSTRUCTION

6.4.1 General

Concrete construction should follow the UBC and ACI guidelines regarding design, mix placement and curing of the concrete. If desired, we could provide quality control testing of the concrete during construction.

6.4.2 Cement Type

The sulfate content was determined in the laboratory for a representative onsite soil sample. The results indicate that the water soluble sulfate is approximately 0.04 percent by weight, which is considered negligible as per Table 19-A-4 of the UBC. Based upon the test results, type II cement or an equivalent may be used in those concrete elements that will be in contact with the upper soils.

6.4.3 Concrete Flatwork

Exterior concrete flatwork (patios, walkways, driveways, etc.) is often some of the most visible aspects of site development. They are typically given the least level of quality control, being considered "non-structural" components. Cracking of these features is fairly common due to various factors. While cracking is not usually detrimental, it is unsightly. We suggest that the same standards of care be applied to these features as to the structure itself.

One of the simplest means to control cracking is to provide weakened joints for cracking to occur along. These do not prevent cracks from developing; they simply provide a relief point for the stresses that develop. These joints are widely accepted means to control cracks but are not always effective. However, control joints are more effective the more closely spaced they are. Therefore, we recommended that control joints be provided in accordance with ACI Guidelines.

Other methods to control cracking in the slab include careful control of water/cement ratios in the concrete, along with taking appropriate curing precautions during the placement of concrete in hot or windy weather.

6.5 RETAINING WALL DESIGN AND CONSTRUCTION

6.5.1 General Design Criteria

Recommendations presented herein may apply to typical masonry or concrete vertical retaining walls to a maximum height of 10 feet. Additional review and recommendations should be requested for higher walls.

Foundations for retaining walls embedded a minimum of 18 inches into compacted fill should be designed using a net allowable bearing capacity of 2,000 psf. An increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads). The passive earth pressure may be computed as an equivalent fluid having a density of 250 psf per foot of depth, to a maximum earth pressure of 3,000 psf. A coefficient of friction between soil and concrete of 0.30 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

An equivalent fluid pressure approach may be used to compute the horizontal active pressure against the wall. The appropriate fluid unit weights are given in Table 6.5.1 below for specific slope gradients of retained materials.

TABLE 6.5.1 – ACTIVE EARTH PRESSURES

Surface Slope of Retained Materials (H:V)	Equivalent Fluid Pressure (PCF)
Level	35
2:1	55

The above equivalent fluid weights do not include other superimposed loading conditions such as expansive soil, vehicular traffic, structures, seismic conditions or adverse geologic conditions.

6.5.2 Wall Backfill and Drainage

The onsite sandy materials possessing a low expansion potential are suitable for backfill provided they are screened of greater than 3-inch size gravels. Presence of other materials might necessitate revision to the parameters provided and modification of wall designs. The backfill materials should be placed in lifts no greater than 8-inches in thickness and compacted at 90 percent relative compaction in accordance with ASTM Test Method D1557-00. Proper surface drainage needs to be provided and maintained.

Retaining walls should be provided with an adequate pipe and gravel back drain system to prevent build up of hydrostatic pressures. Backdrains should consist of a 4-inch diameter perforated PVC pipe embedded in a minimum of one cubic foot per lineal foot of 3/8 to one inch clean crushed rock or equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. A minimum of two outlets should be provided for each drain section.

Walls from 2 to 4 feet in height may be drained using localized gravel packs behind weep holes at 10 feet maximum spacing (e.g. approximately 1.5 cubic feet of gravel in a woven plastic bag). Weep holes should be provided or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may still collect in front of wall.

6.6 POST CONSTRUCTION GUIDELINES

6.6.1 Landscape Maintenance and Planting

Water has been shown to weaken the inherent strength of soil, and slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from graded slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Controlling surface drainage and runoff, and maintaining a suitable vegetation cover can reduce erosion. Plants selected for landscaping should be lightweight, deep-rooted types that require little water and are capable of surviving the prevailing climate.

Over watering should be avoided. The soils should be maintained in a solid to semi-solid state as defined by the materials' Atterberg Limits. Care should be taken when adding soil amendments to avoid excessive watering. Leaching as a method of soil preparation prior to planting is not recommended.

An abatement program to control ground-burrowing rodents should be implemented and maintained. This is critical as burrowing rodents can decreased the long-term performance of slopes.

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundation. This type of landscaping should be avoided. If used, then extreme care should be exercised with regard to the irrigation and drainage in these areas. Waterproofing of the foundation and/or subdrains may be warranted and advisable. We could discuss these issues, if desired, when plans are made available.

6.6.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground. Pad drainage should be directed toward approved area(s).

Positive drainage should not be blocked by other improvements. Even apparently minor changes or modifications can cause problems.

It is the owner's responsibility to maintain and clean drainage devices on or contiguous to their lot. In order to be effective, maintenance should be conducted on a regular and routine schedule and necessary corrections made prior to each rainy season.

As discussed previously, groundwater was encountered in each of our exploratory borings at approximate depths of between 9 and 15 feet below existing site grades. Consequently, it appears that the implementation of a de-watering system might be warranted, if below-grade construction (i.e. basements, etc.) is planned to extend down to or below these depths.

Variations in the rate/consistency of de-watering procedures may have direct influences on the observed static groundwater levels at the site and adjacent areas. As such, we recommend the implementation and operation (as deemed necessary) of de-watering procedures/equipment both during subterranean construction (if planned) and throughout the lifetime of the structure(s). We recommend that a contractor specializing in the design and implementation of de-watering systems be consulted prior to the beginning of construction activities.

6.7 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that site grading, specifications, and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. We also recommend that GeoTek representatives be present during site grading and foundation construction to check for proper implementation of the geotechnical recommendations. These representatives should perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of on-site and import materials for fill placement, and collect soil samples for laboratory testing where necessary.
- Observe the fill for uniformity during placement including utility trenches. Also, test the fill for field density and relative compaction.
- Observe and probe foundation materials to confirm suitability of bearing materials and proper footing dimensions.

If requested, GeoTek will provide a construction observation and compaction report to comply with the requirements of the governmental agencies having jurisdiction over the project. We recommend that these agencies be notified prior to commencement of construction so that necessary grading permits can be obtained.

7. LIMITATIONS

The materials observed on the project site appear to be representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during site construction. Site conditions may vary due to seasonal changes or other factors. GeoTek, Inc. assumes no responsibility or liability for work, testing or recommendations performed or provided by others.

Since our recommendations are based the site conditions observed and encountered, and laboratory testing, our conclusion and recommendations are professional opinions that are limited to the extent of the available data. Observations during construction are important to allow for any change in recommendations found to be warranted. These opinions have been derived in accordance with current standards of practice and no warranty is expressed or implied. Standards of practice are subject to change with time.

8. SELECTED REFERENCES

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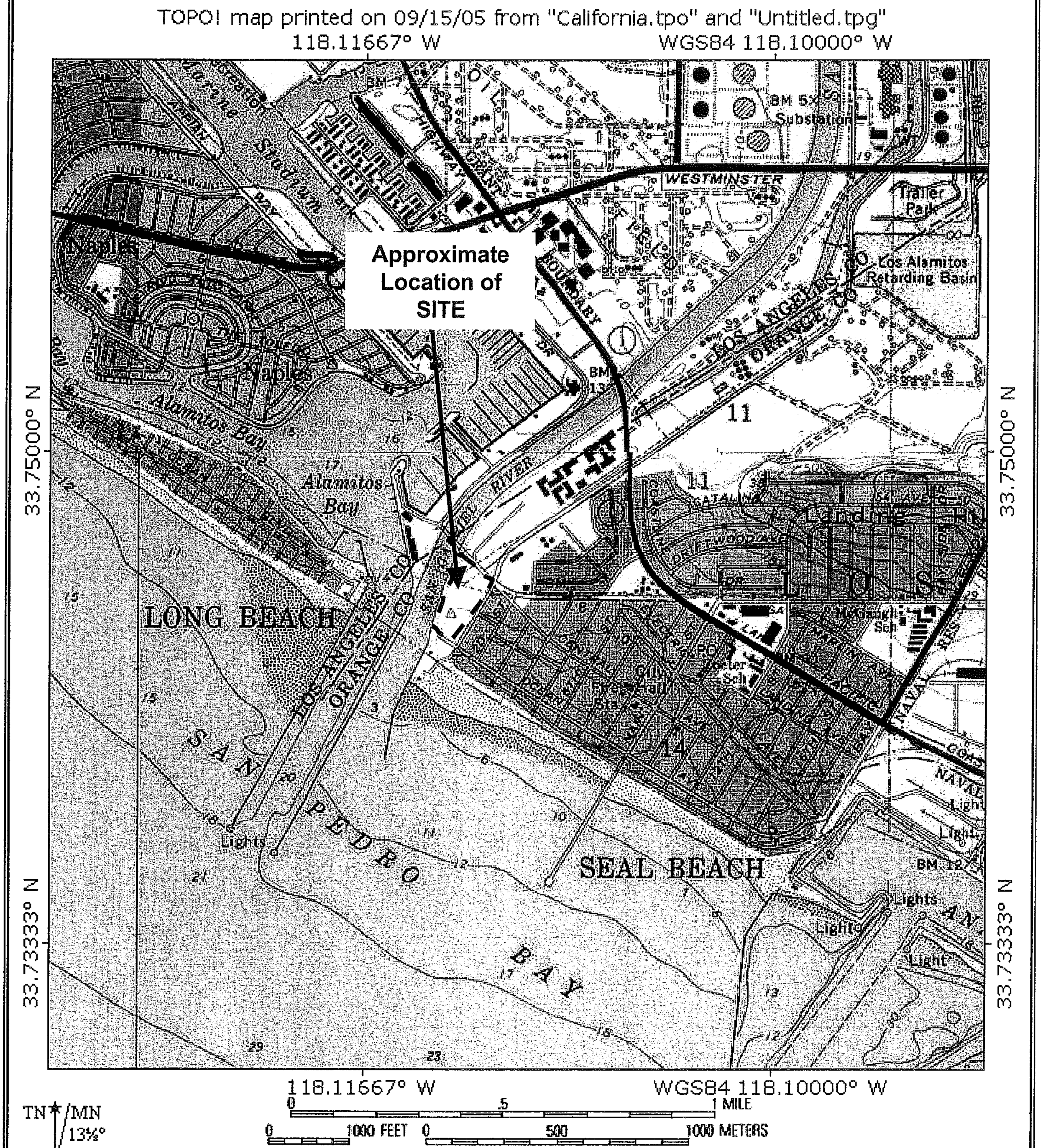


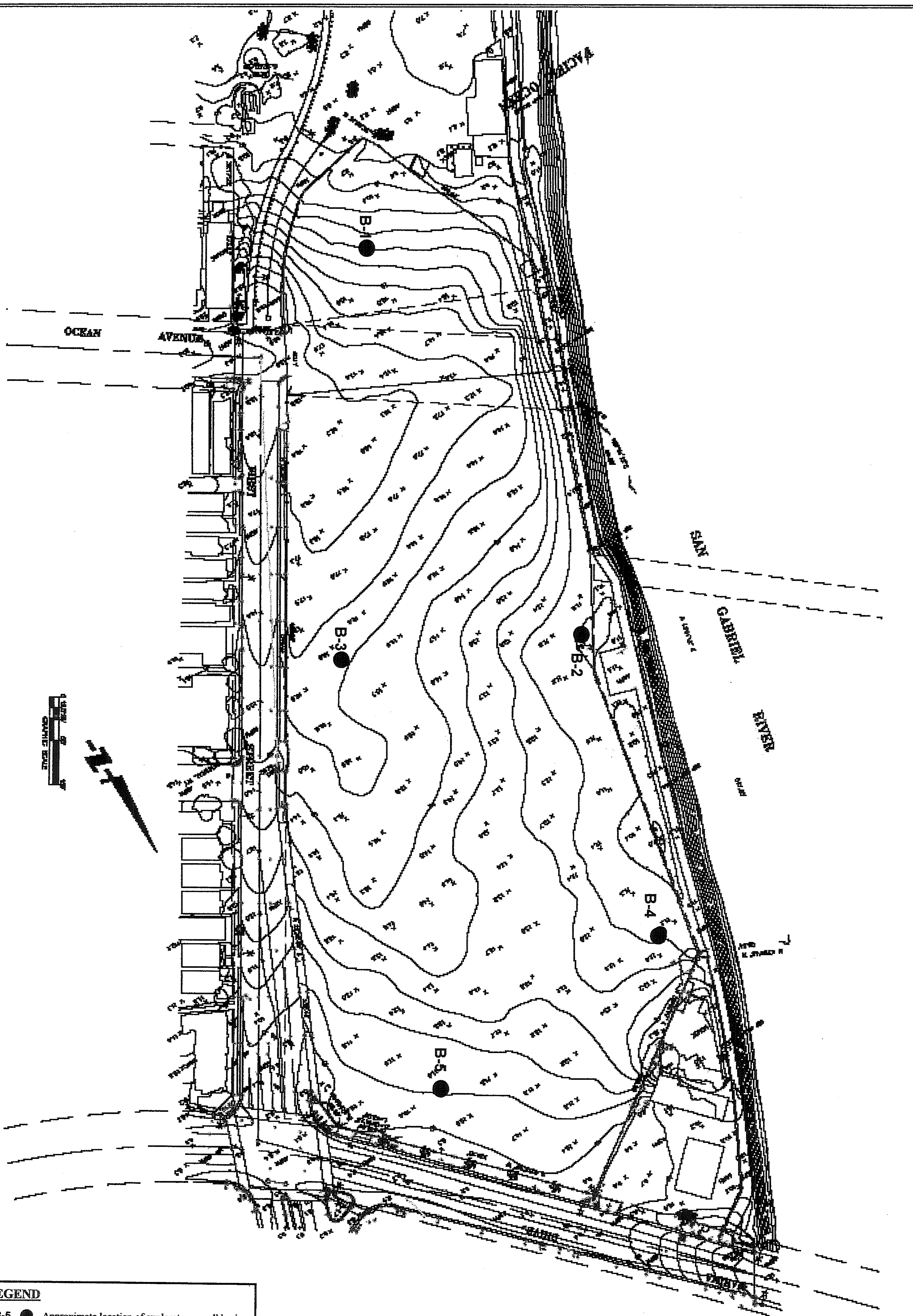
CELEBRATE HOMES
Seal Beach
Seal Beach, California

GeoTek Project Number: 2888SD3

USGS 7.5-Minute Topographic Map

Figure 1
Site Location Map

GEO TEK, INC.
1384 Poinsettia Avenue, Suite A
Vista, California 92081-8505



LEGEND

B-5 ● Approximate location of exploratory small borings

Note: :Map prepared by Coast Surveying, Inc. dated November 11, 2002.

CELEBRATE HOMES **Seal Beach** Seal Beach, California	**Figure 2** **Boring** **Location** **Plan**	GEO TEK, INC. 1384 Poinsettia Avenue, Suite A Vista, California 92081
GeoTek PN: 2888SD3 September 2005		

APPENDIX A

LOGS OF EXPLORATORY BORINGS

BORINGS B-1 THROUGH B-5

PROPOSED RESIDENTIAL DEVELOPMENT

SEAL BEACH, ORANGE COUNTY, CALIFORNIA

PROJECT NO.: 2888SD3

GEO TEK

A - FIELD TESTING AND SAMPLING PROCEDURES

The Standard Penetration Test (SPT)
The SPT is performed in accordance with ASTM Test Method D 1586-99. The SPT sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. The split-barrel sampler has an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The samples of earth materials collected in the sampler are typically classified in the field, bagged, sealed and transported to the laboratory for further testing.

The Modified Split-Barrel Sampler (Ring)
The Ring sampler is driven into the ground in accordance with ASTM Test Method D 3550-84. The sampler, with an external diameter of 3.0 inches, is lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. The samples are removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

Bulk Samples (Large)
These samples are normally large bags of representative earth materials over 20 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

Bulk Samples (Small)
These are plastic bags samples which are normally airtight and contain less than 5 pounds in weight of representative earth materials collected from the field by means of hand digging or exploratory cuttings. These samples are primarily used for determining natural moisture content and classification indices.

B - BORING LOG LEGEND

The following abbreviations and symbols often appear in the classification and description of soil and rock on the logs of borings:

SOILS

USCS	Unified Soil Classification System
f-c	Fine to coarse
f-m	Fine to medium

GEOLOGIC

B: Attitudes	Bedding: strike/dip
J: Attitudes	Joint: strike/dip
C:	Contact line
....	Dashed line denotes USCS material change
———	Solid Line denotes unit / formational change
━━━	Thick solid line denotes end of boring

(Additional denotations and symbols are provided on the logs of borings)

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Celebrate Homes	**DRILLER:**	Layne Christensen Company	**LOGGED BY:**	LG
PROJECT NAME:	Seal Beach	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	Armandon
PROJECT NO.:	2888SD3	**HAMMER:**	140lbs/30in - Auto	**RIG TYPE:**	CME 75
LOCATION:	See Site Plan	**ELEVATION:**	±14 feet	**DATE:**	8/22/2005

BORING NO.: *B-1*

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
	Small Bulk		B1-1	SM	Fill Gray-brown, dry, loose, silty fine to medium SAND; with gravel ; rootlets @1': becomes yellow-brown, damp, silty fine SAND ; trace roots			
	Ring	3 5 8	B1-2	ML	Light yellow-brown, damp, stiff, clayey SILT	18	94	
5	SPT	2 2 4	B1-3		-same- interbedded with red fine to medium sand	19		EI=54
				SM/ML	Yellow-brown, moist, medium dense, silty fine SAND to sandy SILT			
	Ring	3 14		SP	Yellow, moist, medium dense, fine SAND with silt			
		16	B1-4	SP-SM	Alluvium Gray, moist, medium dense, fine SAND with silt	4	98	
10	SPT	3 7 11	B1-5	SP	Gray, wet, medium dense, fine SAND with silt	Water Table		
	Ring	5 31 26	B1-6		Interbedded with light gray & yellow -brown, moist, medium dense, clayey SILT; with red-orange-yellow fine to medium SAND	20	106	
15	SPT	6	B1-7A	ML	Gray, red, wet, firm clayey SILT; black staining; iron oxide staining	20		
		20 25	B1-7	SP-SM	Brown, orange, wet, medium dense, fine SAND to silty fine SAND	24		
20	Ring	7 18 46	B1-8	SP	Brown, dense, wet to saturated, fine to medium SAND; interbedded with gray-orange-red, clayey silt	18	87	
25	SPT	2 14 36	B1-9		@25'; becomes gray, saturated, dense, fine to medium SAND with silt; interbedded with silty fine sand	24		
30					**(continued)**			

LEGEND

Sample type: —Ring —SPT —Small Bulk —Large Bulk —No Recovery —Water Table

Lab testing: AL = Atterberg Limits; SR = Sulfate/Resisitivity Test; EI = Expansion Index; SH = Shear Test; SA = Sieve Analysis; CO = Consolidation test; RV = R-Value Test; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Celebrate Homes	DRILLER:	Layne Christensen Company	LOGGED BY:	LG
PROJECT NAME:	Seal Beach	DRILL METHOD:	8" Hollow Stem Auger	OPERATOR:	Armandon
PROJECT NO.:	2888SD3	HAMMER:	140lbs/30in - Auto	RIG TYPE:	CME 75
LOCATION:	See Site Plan	ELEVATION:	± 14 feet	DATE:	8/22/2005

BORING NO.: B-1

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					(continued)			
30	SPT	4 10 20	B1-10B B1-10A B1-10	SP	Yellow-brown, saturated, medium dense, fine to medium SAND with silt interbedded with gray, red, very stiff, clayey SILT and gray-brown, fine to medium SAND with silt & gravel			
35	SPT	8 10 25	B1-11	SP-SM	Gray-brown, saturated, dense, fine to medium SAND with silt to silty fine to medium SAND	23		
40	SPT	18 44 50/5"	B1-12		@40': Gray-brown, saturated, very dense, fine to medium SAND with silt to silty fine to medium SAND	19		
45	SPT	2 5 10	B1-13	SP-SM	Gray-brown, saturated, medium dense, fine to medium SAND to silty fine to medium SAND	20		
50	SPT	3 9 21	B1-14	SP	Gray-brown, saturated, medium dense, fine to medium SAND; trace fine gravel, shell fragments			
55	SPT	4 17 29	B1-15		-same; becomes dense			
60					-HOLE TERMINATED AT 59 FEET- Hole backfilled with bentonite & cement Groundwater encountered at 12 feet			

LEGEND

Sample type: —Ring —SPT —Small Bulk —Large Bulk —No Recovery —Water Table

Lab testing:
AL = Atterberg Limits
EI = Expansion Index
SA = Sieve Analysis
RV = R-Value Test
SR = Sulfate/Resisitivity Test
SH = Shear Test
CO = Consolidation test
MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Celebrate Homes	**DRILLER:** Layne Christensen Company	**LOGGED BY:** LG
PROJECT NAME: Seal Beach	**DRILL METHOD:** 8" Hollow Stem Auger	**OPERATOR:** Armandon
PROJECT NO.: 2888SD3	**HAMMER:** 140lbs/30in - Auto	**RIG TYPE:** CME 75
LOCATION: See Site Plan	**ELEVATION:** ± 11.5 feet	**DATE:** 8/22/2005

BORING NO.: B-2

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
	Large Bulk		B2-1	SM	<u>Fill</u> Light-brown, dry, loose, silty fine to medium SAND; with gravel ; rootlets			MD=131@8.5% SH=28.4°
	SPT	10 15 17	B2-2		@1.5': becomes light brown, moist, dense, silty fine to medium SAND with gravel; calcium carbonate; black staining	7		
5	Ring	10 28 30/1"	B2-3		-same-with concrete pieces; shell fragments; gravel @6': rocks; becomes dark brown, moist, loose, silty fine to medium SAND with gravel	9	116	
	SPT	27 11 8	B2-4			10		
10	No Recovery	3 1 1		SM	Dark gray, saturated, very loose, silty fine SAND with clay @12': rock	▽		
	SPT	3 17 50/4"	B2-5		-same			
15	No Recovery / Small Bulk	50/4"	B2-6 B2-6A	SW	@15': becomes dark gray, saturated, fine to coarse SAND			
	SPT	2 2 3	B2-7	ML	Dark gray-black, saturated, loose, sandy SILT			
20								
	No Recovery / Small Bulk	2 30/5"	B2-8	CL-ML	Gray, saturated, soft, silty CLAY to clayey SILT with rocks sampler bouncing on rocks			
					-HOLE TERMINATED AT 23 FEET- Hole backfilled with bentonite & cement Groundwater encountered at 10 feet			
25								
30								

LEGEND

Sample type: —Ring —SPT —Small Bulk —Large Bulk —No Recovery ▽ —Water Table

Lab testing:
AL = Atterberg Limits
SR = Sulfate/Resisitivity Test
EI = Expansion Index
SH = Shear Test
SA = Sieve Analysis
CO = Consolidation test
RV = R-Value Test
MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Celebrate Homes	**DRILLER:** Layne Christensen Company	**LOGGED BY:** LG
PROJECT NAME: Seal Beach	**DRILL METHOD:** 8" Hollow Stem Auger	**OPERATOR:** Armandon
PROJECT NO.: 2888SD3	**HAMMER:** 140lbs/30in - Auto	**RIG TYPE:** CME 75
LOCATION: See Site Plan	**ELEVATION:** ± 15.feet	**DATE:** 8/22/2005

BORING NO.: *B-3*

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
	Small Bulk		B3-1	SM	**Fill** Gray-brown, dry, loose, silty fine to medium SAND; with gravel ; rootlets			
	Ring	5 21 30	B3-2		**Alluvium** Red-brown, moist, medium dense, silty fine to medium SAND; calcium carbonate; black staining	9	130	
5	SPT	3 12 16	B3-3		-same	10		
	Ring	12 30 46	B3-4	SC	Red-brown, moist, dense, clayey fine to coarse SAND; black staining	9	130	
10	SPT	5 8 16	B3-5		-same, becomes wet, medium dense	24		
	Small Bulk		B3-6A	ML	Light brown, wet, silty fine SAND ot sandy SILT ; interbedded with silty clay	29	110	
15	SPT	50/2"	B3-6	SM	Red, gray, orange, wet, medium dense, silty fine to medium SAND; iron oxide staining			
20	SPT	12 22 28	B3-7	SP-SM	Gray, saturated, dense, fine to medium SAND to silty fine to medium SAND	25		
25	Ring	11		SW	Gray, moist, medium dense, fine to coarse SAND			
	Ring	20 20	B3-8	ML	Light brown, moist, dense, clayey SILT; iron oxide	18	108	
					-HOLE TERMINATED AT 26.5 FEET- Hole backfilled with bentonite & cement Groundwater encountered at 15 feet			
30								

LEGEND

Sample type: Ring — SPT — Small Bulk — Large Bulk — No Recovery — Water Table

Lab testing: AL = Atterberg Limits; SR = Sulfate/Resisitivity Test; EI = Expansion Index; SH = Shear Test; SA = Sieve Analysis; CO = Consolidation test; RV = R-Value Test; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Celebrate Homes	DRILLER:	Layne Christensen Company	LOGGED BY:	LG
PROJECT NAME:	Seal Beach	DRILL METHOD:	8" Hollow Stem Auger	OPERATOR:	Armandon
PROJECT NO.:	2888SD3	HAMMER:	140lbs/30in - Auto	RIG TYPE:	CME 75
LOCATION:	See Site Plan	ELEVATION:	± 10.5 feet	DATE:	8/22/2005

BORING NO.: B-4

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
	Small Bulk		B4-1	SM	**Fill** Red-brown, dry, silty fine to medium SAND with clay & gravel			
	Ring	5 9 13	B4-2	ML	Red-brown, moist, stiff, clayey SILT	22		
5	Small Bulk		B4-2A					
	SPT	4 4 5	B4-3		@5': becomes brown-gray, moist, stiff, clayey SILT; with root hairs, trace pinholes, interbedded seams sand	25		
	Ring	4 5 7	B4-4A B4-4		@8': becomes wet	29	94	
10	SPT	4 5 5	B4-5A B4-5	SP	Dark gray, wet, loose, fine SAND -same becomes gray and with shell fragments	23		
	Ring	3 3 3	B4-6A B4-6	ML	Light gray, wet, medium stiff, clayey SILT @13': becomes dark gray, saturated, loose, sandy SILT with clay; micaceous; shell fragments; rootlets	32		
15	SPT	1 3 3	B4-7		@15': becomes dark gray, saturated, loose, sandy SILT with clay; rootlet @16': becomes gray-green, saturated, medium stiff, clayey SILT ; trace roots	24		
	Ring	4 6 6	B4-8		@18': becomes gray, saturated, stiff, clayey SILT with sand	29	95	
20	SPT	2 3 6	B4-9		@20': becomes gray, saturated, loose, sandy SILT with clay; micaceous @21': Gray-green, saturated, stiff, clayey SILT to silty CLAY with brown, clay seams; organics; roots; oxide spotting			
25	Ring	6 7 7	B4-10A B4-10	ML	Light gray, saturated, stiff, clayey SILT with sand; roots	26	96	
				SM	Gary, wet, loose, silty fine SAND			
30					**(continued)**			

Water table symbol at approximately 9 ft (Water Content column).

LEGEND

Sample type: —Ring —SPT —Small Bulk —Large Bulk —No Recovery —Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; CO = Consolidation test; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Celebrate Homes	**DRILLER:**	Layne Christensen Company	**LOGGED BY:**	LG
PROJECT NAME:	Seal Beach	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	Armandon
PROJECT NO.:	2888SD3	**HAMMER:**	140lbs/30in - Auto	**RIG TYPE:**	CME 75
LOCATION:	See Site Plan	**ELEVATION:**	± 10.5 feet	**DATE:**	8/22/2005

BORING NO.: B-4

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					(continued)			
30	SPT	3 4 8	B4-11A B4-11	SP	Gray, saturated, medium dense, fine to medium SAND interbedded with clayey SILT			
35	SPT	9		ML	Gray, saturated, stiff, clayey SILT			
		25 26	B4-12	SM	**Alluvium** Yellow-green, moist, very dense, silty fine SAND with clay			
40	SPT	5	B4-13B		-same			
		8 10	B4-13A B4-13	CL	Yellow-brown, moist, very stiff, silty CLAY Interbedded with Red-brown, moist, very stiff, clayey SILT with sand			
45	SPT	7	B4-14A		-same			
		15 26	B4-14	SM	Yellow-brown, red, gray, moist, dense, silty fine SAND			
50	SPT	23 25 33	B4-15	SP	Gray, wet, very dense, fine SAND			
55					-HOLE TERMINATED AT 51.5 FEET- Hole backfilled with bentonite & cement Groundwater encountered at 9 feet			
60								

LEGEND

Sample type: —Ring —SPT —Small Bulk —Large Bulk —No Recovery —Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	CO = Consolidation test	MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Celebrate Homes	DRILLER:	Layne Christensen Company	LOGGED BY:	LG
PROJECT NAME:	Seal Beach	DRILL METHOD:	8" Hollow Stem Auger	OPERATOR:	Armandon
PROJECT NO.:	2888SD3	HAMMER:	140lbs/30in - Auto	RIG TYPE:	CME 75
LOCATION:	See Site Plan	ELEVATION:	± 13 feet	DATE:	8/22/2005

BORING NO.: B-5

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					Fill			
				SM	Red-brown, damp, loose, silty fine SAND with gravel			
	SPT	16 16 16	B5-1		@2': becomes red-brown, moist, dense, silty fine SAND; trace roots; shell fragments interbedded with sandy SILT			
5	Ring	7 6 7	B5-2	SM-SP	-same @5': becomes brown, moist, loose, fine SAND with shell fragments; micaceous			
				SM	Dark gray, green, moist to wet, loose, silty fine SAND ; micaceous			
	SPT	2 1 1	B5-3		@7.5': becomes gray-black, saturated, very loose, silty fine SAND interbedded with clayey silt	▽		
10	Large Bulk		B5-4	ML-CL	Gray, wet to saturated, medium stiff, clayey SILT to firm silty CLAY			
	Ring	6 7 11	B5-5A B5-5	ML	Light gray, moist, loose, sandy SILT	28		
	SPT	1 1 1	B5-6A B5-6	ML	Light gray, wet, very loose, sandy SILT trace clay	28		
15	Ring	4 5 10	B5-7		@15': becomes gray & green, moist, loose, sandy SILT; micaceous; seashell fragments	33	88	
20	SPT	3 3 7	B5-8	SW-SM	Gray, saturated, loose, silty fine to coarse SAND with abundance of shell fragments: interbedded with silty fine to medium SAND	21		
25	SPT	2 3 5	B5-9		-same			
				ML	Dark gray, wet, loose, sandy SILT with clay	33		
30					**(continued)**			

LEGEND

Sample type: —Ring —SPT —Small Bulk —Large Bulk —No Recovery ▽ —Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	CO = Consolidation test	MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Celebrate Homes	DRILLER:	Layne Christensen Company	LOGGED BY:	LG
PROJECT NAME:	Seal Beach	DRILL METHOD:	8" Hollow Stem Auger	OPERATOR:	Armandon
PROJECT NO.:	2888SD3	HAMMER:	140lbs/30in - Auto	RIG TYPE:	CME 75
LOCATION:	See Site Plan	ELEVATION:	± 13 feet	DATE:	8/22/2005

BORING NO.: *B-5*

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					(continued)			
30	SPT	2 4 4	B5-10	ML	Dark gray, saturated, loose, sandy SILT; trace clay			
35	SPT	5 10	B5-11B B5-11A		@35': becomes gray-black & green, moist to wet, medium dense, sandy SILT interbedded with coarse sand			
		25	B5-11	SM	**Alluvium** Green-yellow, moist, dense, silty fine SAND			
					-HOLE TERMINATED AT 36.5 FEET- Hole backfilled with bentonite & cement Groundwater encountered at 9 feet			
40								
45								
50								
55								
60								

LEGEND

Sample type: —Ring —SPT —Small Bulk —Large Bulk —No Recovery —Water Table

Lab testing:
AL = Atterberg Limits
SR = Sulfate/Resisitivity Test
EI = Expansion Index
SH = Shear Test
SA = Sieve Analysis
CO = Consolidation test
RV = R-Value Test
MD = Maximum Density

APPENDIX B

RESULTS OF LABORATORY TESTING

PROPOSED RESIDENTIAL DEVELOPMENT

SEAL BEACH, ORANGE COUNTY, CALIFORNIA

PROJECT NO.: 2888SD3

SUMMARY OF LABORATORY TESTING

Classification
Soils were classified visually according to the Unified Soil Classification System (ASTM Test Method D2487). The soil classifications are shown on the logs of exploratory borings in Appendix A.

Grain size distribution (particle size analysis) was performed on selected samples in general accordance with ASTM D422. Results of the grain size analysis are included herein (see Plates SA-1 through SA-6).

Liquid limit, plastic limit and plasticity index were determined in general accordance with ASTM Test Method D4318. Results are shown on the logs of exploratory borings in Appendix A.

In Situ Moisture and Unit Weight
The field moisture content was measured in the laboratory on selected samples collected during the field investigation. The field moisture content is determined as a percentage of the dry unit weight. The dry density was measured in the laboratory on selected ring samples. The results are shown on the logs of exploratory borings in Appendix A.

Expansion Index
Expansion Index testing was performed on representative soil samples. Testing was performed in general accordance with ASTM Test Method D4829. The Expansion Index (EI) test results indicate a medium expansion potential. The results are presented on the logs of exploratory borings in Appendix A.

Sulfate Content
Analysis to determine the water-soluble sulfate content was performed in general accordance with California Test No. 417. Results of the testing indicated a sulfate content of .04%, which is considered negligible as per Table 19-A-4 of the UBC.

Resistivity
Representative surficial soil samples were collected and tested for their pH and minimum resistivity in general accordance with California Test 643. The results of the testing are included herein (see Plates SR-1).

Direct Shear
Shear testing was performed in a direct shear machine of the strain-control type in general accordance with ASTM Test Method D3080. The rate of deformation is 0.03 inches per minute. The sample was sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. The shear test results are presented on Plate SH-1 included herein.

GeoTek, Inc.

EXPANSION INDEX TEST

(ASTM D4829)

Project Name: Seal Beach
Project Number: 2888-SD3
Project Location: B1 @ 3 - 6'

Tested/ Checked By: AS Lab No 1903
Date Tested: 8/30/2005
Sample Source: B1 @ 3 - 6'
Sample Description:

Ring Id 12 Ring Dia. " 4" Ring I 1"
Loading weight: 5516. grams

DENSITY DETERMINATION

A	Weight of compacted sample & ring	740.5
B	Weight of ring	370
C	Net weight of sample	370.5
D	Wet Density, lb / ft3 (C*0.3016)	111.7
E	Dry Density, lb / ft3 (D/1.F)	**104.3**

SATURATION DETERMINATION

F	Moisture Content, %	7.1
G	(E*F)	740.8
H	(E/167.232)	0.62
I	(1.-H)	0.38
J	(62.4*I)	23.5
K	(G/J)= L % Saturation	**31.6**

READINGS

DATE	TIME	READING	
8/30/2005	2:40	0.14	Initial
8/30/2005	2:50	0.138	10 min/Dry
8/30/2005	2:51	0.145	1 min/Wet
8/30/2005	2:56	0.152	5 min/Wet
8/30/2005	4:00	0.171	Random
8/31/2005	8:00	0.207	Final

FINAL MOISTURE

Weight of wet sample & tare	Weight of dry sample & tare	Tare	% Moisture
270	245.3	6.7	**10.4%**

EXPANSION INDEX = 54
(@50% SATURATION)

MAXIMUM DENSITY CURVE

Project No.: 2888-SD3

Date: 08/31/05

Project: Seal Beach

Location: B2 @ 0-2'

Elev./Depth: 0-2'

Remarks:

MATERIAL DESCRIPTION

Description: Light Brown Silty Fine Sand

Classifications - **USCS:** **AASHTO:**

Nat. Moist. = **Sp.G. =**

Liquid Limit = **Plasticity Index =**

% > No.4 = % **% < No.200 =**

TEST RESULTS
Maximum dry density = 131 pcf
Optimum moisture = 8.5 %

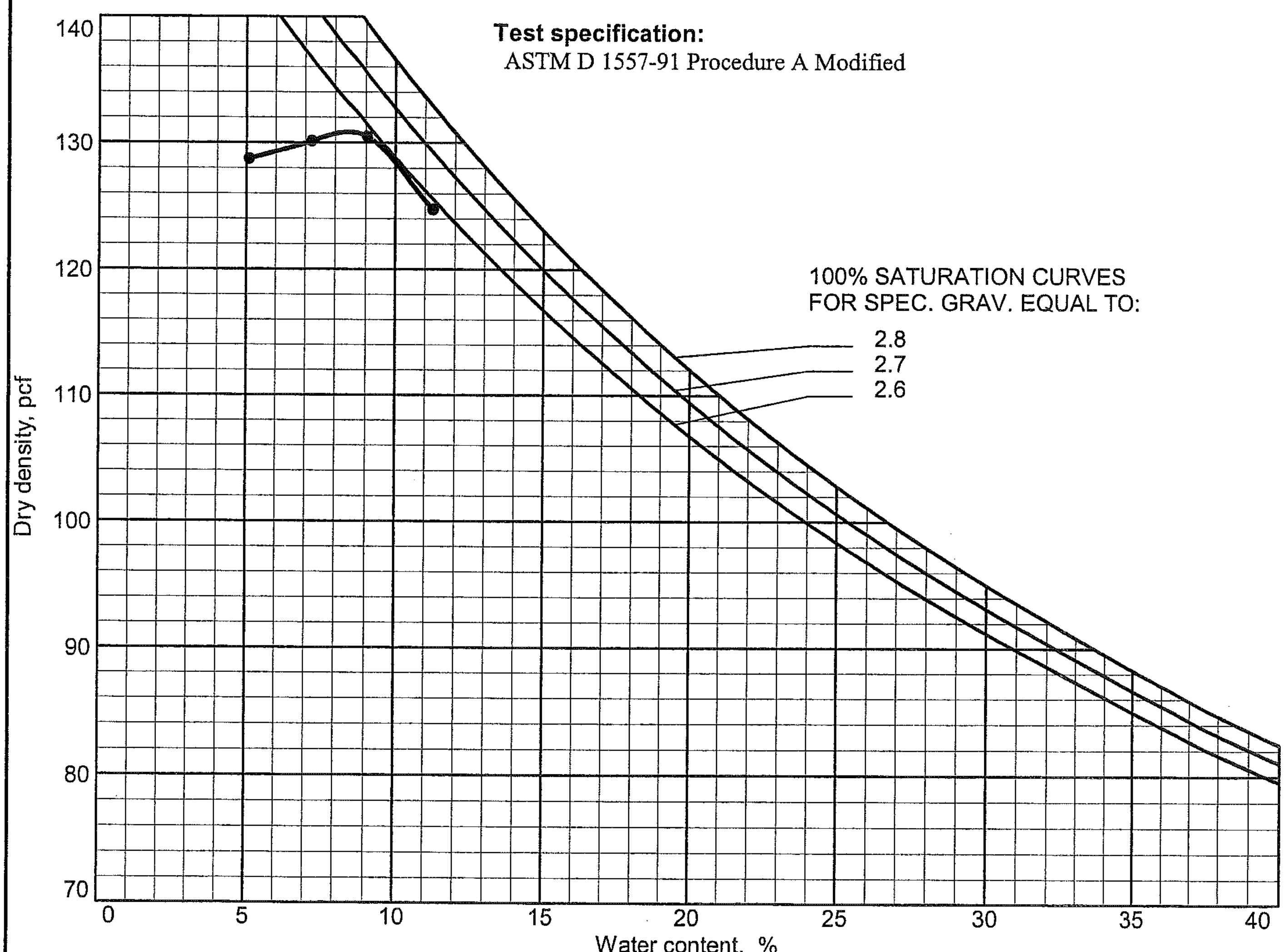
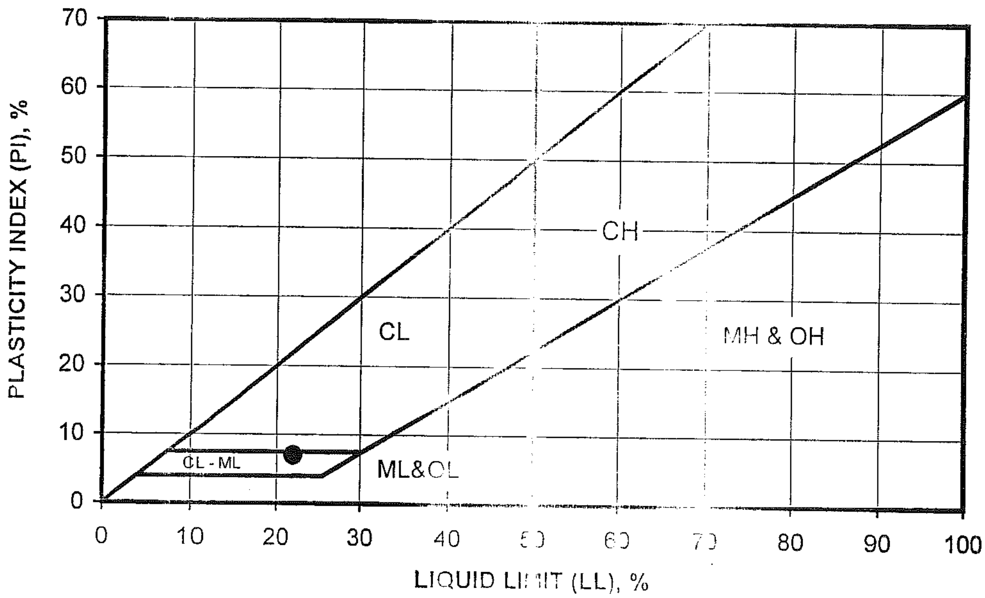


Plate **MD-1**

SYMBOL	SAMPLE No.	BORING / DEPTH (FT)	LL (%)	PL (%)	PI (%)	USCS CLASSIFICATION (Minus No. 40 Sieve Fraction)	REMARKS
●	B1	3 - 6'	22	15	7	CL-ML	



PERFORMED IN GENERAL ACCORDA CE WITH AS 4318-98

GEO TEK, INC.

ATTERBERG LIMITS TEST RESULTS

Seal Bea

FIGURE

PI1

CHECKED BY: DC	FN: LAB
PROJECT NO.: 2888 SD3	DATE: 9/14/05

SIEVE ANALYSIS of COARSE & FINE AGGREGATE

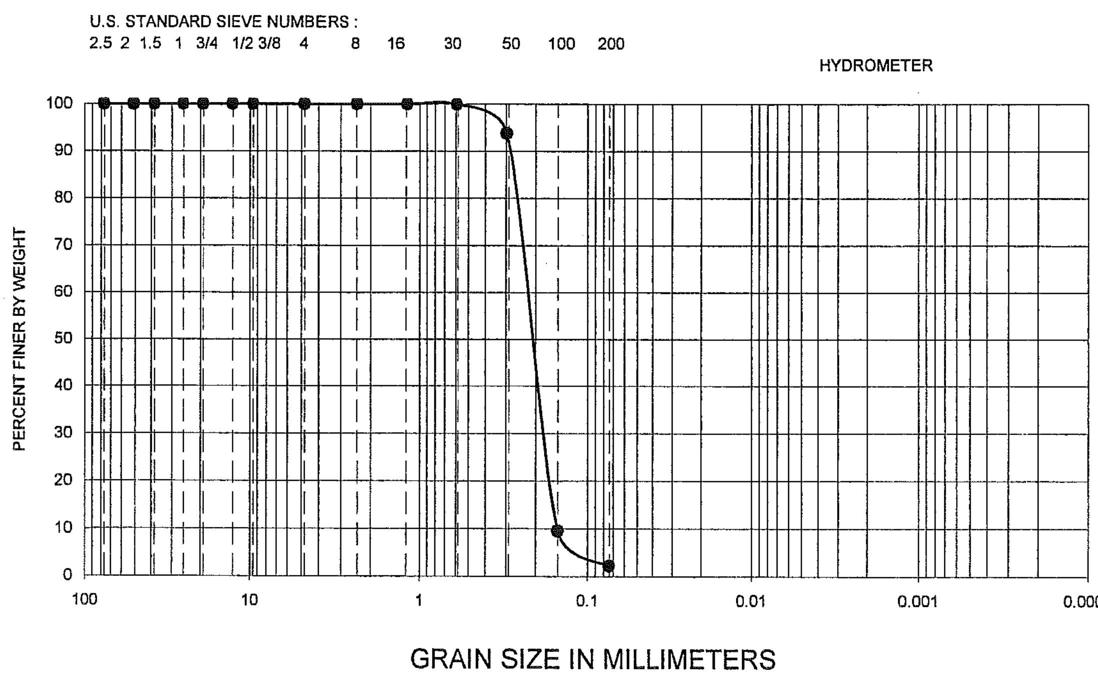
CLIENT:	Celebrate Homes	LAB NO.:	1903
PROJECT:	Seal Beach	PROJECT: NO.:	2888-SD3
MATERIAL LOCATION:	B1 @ 10	DATE:	8.31.05

SAMPLE DESCRIPTION

TOTAL WT. SAMPLE (DRY)	488.3 Dry	**WT. COARSE (+) # 4**	0	Dry	**WT COARSE %**	0.0
Wet Wt. Before Wash (-)#4	488.3 Wet	**WT. FINE (-) # 4**	488.3	Wet	**WT FINE %**	100.0
Dry Wt. Before Wash (-)#4	488.3 Dry		488.3	Dry	**-200%**	2
			0	Moisture Content (- # 4)		0

Sieve Size	WEIGHT RETAINED Ind	WEIGHT RETAINED Cum		% RETAINED Ind	% RETAINED Cum		Combined % Passing	Specs.	
3"/75mm		0			0		100		
2"/50mm		0			0		100		
1.5"/37.5mm		0			0		100		
1"/25mm		0			0		100		
.75"/19mm		0			0		100		
.5"/12.5mm		0			0		100		
.375"/9.5mm		0			0		100		
#4/4.75mm		0			0		100		
#8		0		()	(100)		100		
#16		0.1		()	(100)		100		
#30		0.4		()	(100)		100		
#50		30.4		(6)	(94)		94		
#100		441.3		(90)	(10)		10		
#200		477.5		(98)	(2)		2		
PAN		479.5					0		
WASH		10.8							

Notes:
all weights are in grams



SIEVE ANALYSIS of COARSE & FINE AGGREGATE

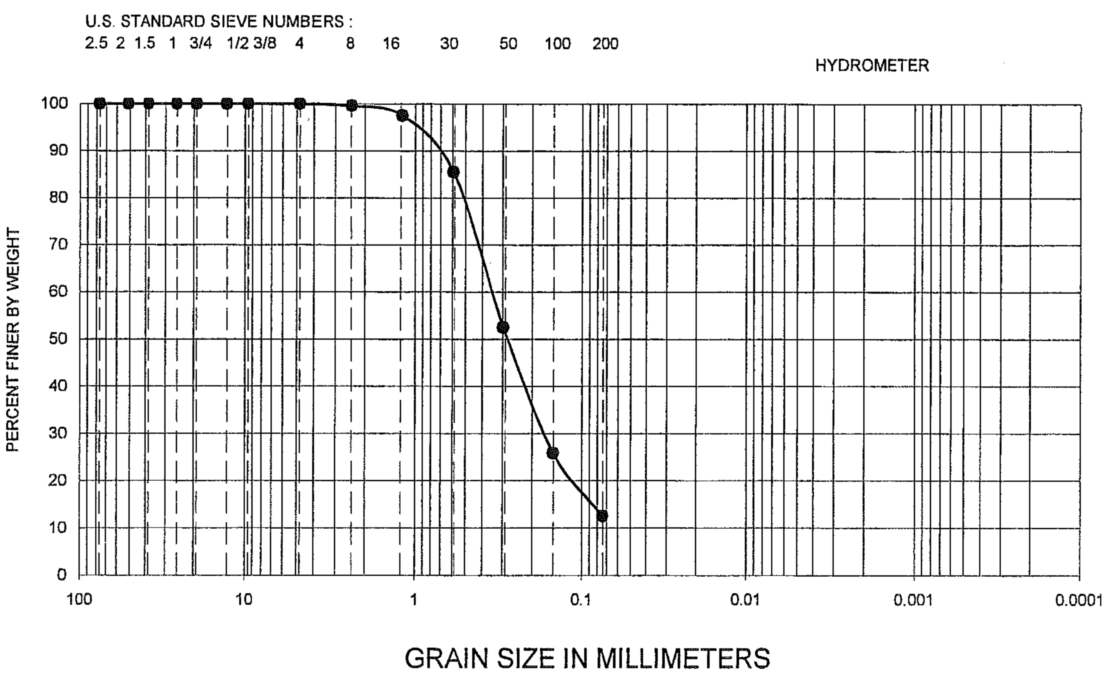
CLIENT:	Celebrate Homes	LAB NO.:	1903
PROJECT:	Seal Beach	PROJECT: NO.:	2888-SD3
MATERIAL LOCATION:	B1 @ 25	DATE:	8.31.05

SAMPLE DESCRIPTION

TOTAL WT. SAMPLE (DRY)	377.3 Dry	**WT. COARSE (+) # 4**	0.1	Dry	**WT COARSE %**	0.0
Wet Wt. Before Wash (-)#4	377.2 Wet	**WT. FINE (-) # 4**	377.2	Wet	**WT FINE %**	100.0
Dry Wt. Before Wash (-)#4	377.2 Dry		377.2	Dry	**-200%**	13
			0	Moisture Content (- # 4)		0

Sieve Size	WEIGHT RETAINED Ind	WEIGHT RETAINED Cum		% RETAINED Ind	% RETAINED Cum		Combined % Passing	Specs.
3"/75mm		0			0		100	
2"/50mm		0			0		100	
1.5"/37.5mm		0			0		100	
1"/25mm		0			0		100	
.75"/19mm		0			0		100	
.5"/12.5mm		0			0		100	
.375"/9.5mm		0			0		100	
#4/4.75mm		0.1			0		100	
#8		1.4		()	(100)		100	
#16		9.2		(2)	(98)		98	
#30		54.5		(14)	(86)		86	
#50		179		(47)	(53)		53	
#100		279.5		(74)	(26)		26	
#200		329.9		(87)	(13)		13	
PAN		337.6					0	
WASH		47.3						

Notes:
all weights are in grams



SIEVE ANALYSIS of COARSE & FINE AGGREGATE

CLIENT: Celebrate Homes | LAB NO.: 1903

PROJECT: Seal Beach | PROJECT: NO.: 2888-SD3

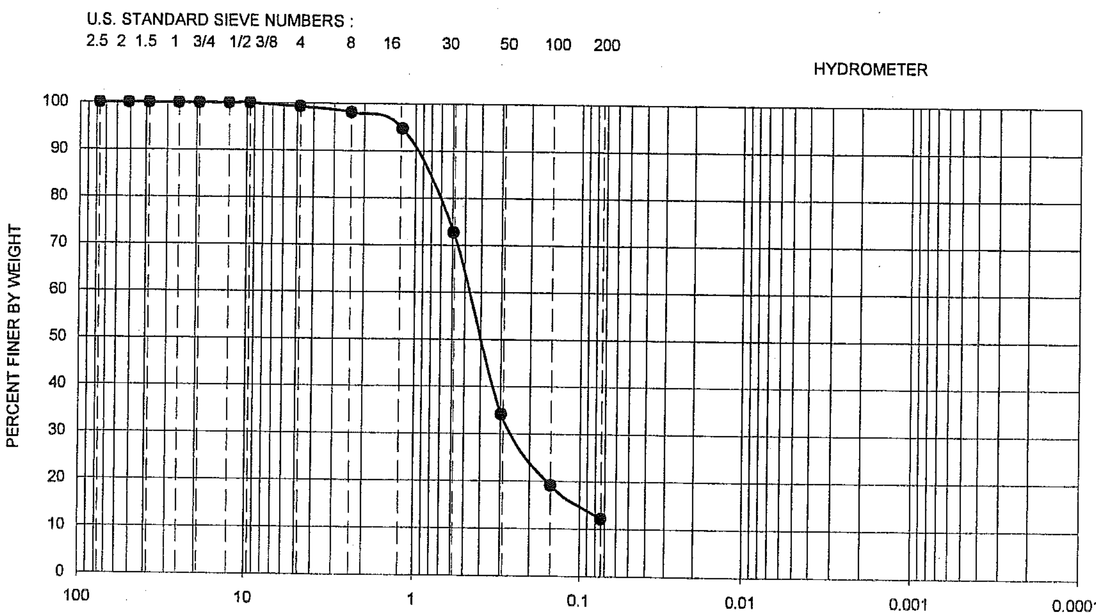
MATERIAL LOCATION: B1 @ 47 | DATE: 8.31.05

SAMPLE DESCRIPTION

TOTAL WT. SAMPLE (DRY)	503.5 Dry	WT. COARSE (+) # 4	3.5	Dry	WT COARSE %	0.7
Wet Wt. Before Wash (-)#4	500.0 Wet	WT. FINE (-) # 4	500.0	Wet	WT FINE %	99.3
Dry Wt. Before Wash (-)#4	500.0 Dry		500.0	Dry	-200%	12
			0	Moisture Content (- # 4)		0

Sieve Size	WEIGHT RETAINED Ind	WEIGHT RETAINED Cum		% RETAINED Ind	% RETAINED Cum		Combined % Passing	Specs.
3"/75mm		0			0		100	
2"/50mm		0			0		100	
1.5"/37.5mm		0			0		100	
1"/25mm		0			0		100	
.75"/19mm		0			0		100	
.5"/12.5mm		0			0		100	
.375"/9.5mm		0			0		100	
#4/4.75mm		3.5			1		99	
#8		5.9		(1)	(99)		98	
#16		22.7		(5)	(95)		95	
#30		133.9		(27)	(73)		73	
#50		327.7		(66)	(34)		34	
#100		403.2		(81)	(19)		19	
#200		438.7		(88)	(12)		12	
PAN		443.3					0	
WASH		61.3						

Notes:
all weights are in grams



GRAIN SIZE IN MILLIMETERS

SIEVE ANALYSIS of COARSE & FINE AGGREGATE

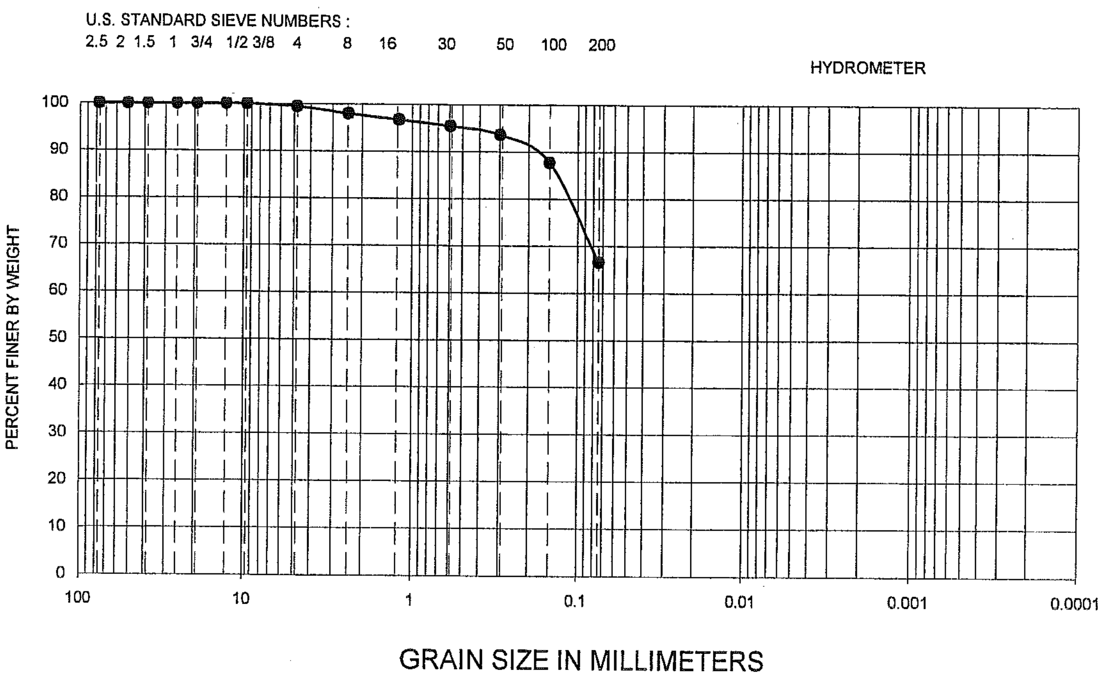
CLIENT:	Celebrate Homes	LAB NO.:	1903
PROJECT:	Seal Beach	PROJECT: NO.:	2888-SD3
MATERIAL LOCATION:	B4 @ 15	DATE:	8.31.05

SAMPLE DESCRIPTION

TOTAL WT. SAMPLE (DRY)	298.9 Dry	**WT. COARSE (+) # 4**	1.6	Dry	**WT COARSE %**	0.5
Wet Wt. Before Wash (-)#4	297.3 Wet	**WT. FINE (-) # 4**	297.3	Wet	**WT FINE %**	99.5
Dry Wt. Before Wash (-)#4	297.3 Dry		297.3	Dry	**-200%**	67
			0	Moisture Content (- # 4)		0

Sieve Size	WEIGHT RETAINED Ind	WEIGHT RETAINED Cum		% RETAINED Ind	% RETAINED Cum		Combined % Passing	Specs.	
3"/75mm		0			0		100		
2"/50mm		0			0		100		
1.5"/37.5mm		0			0		100		
1"/25mm		0			0		100		
.75"/19mm		0			0		100		
.5"/12.5mm		0			0		100		
.375"/9.5mm		0			0		100		
#4/4.75mm		1.6			1		99		
#8		4.4		(1)	(99)		98		
#16		8		(3)	(97)		97		
#30		11.9		(4)	(96)		95		
#50		17.5		(6)	(94)		94		
#100		35.1		(12)	(88)		88		
#200		98.3		(33)	(67)		67		
PAN		106.2					0		
WASH		199.0							

Notes:
all weights are in grams



SIEVE ANALYSIS of COARSE & FINE AGGREGATE

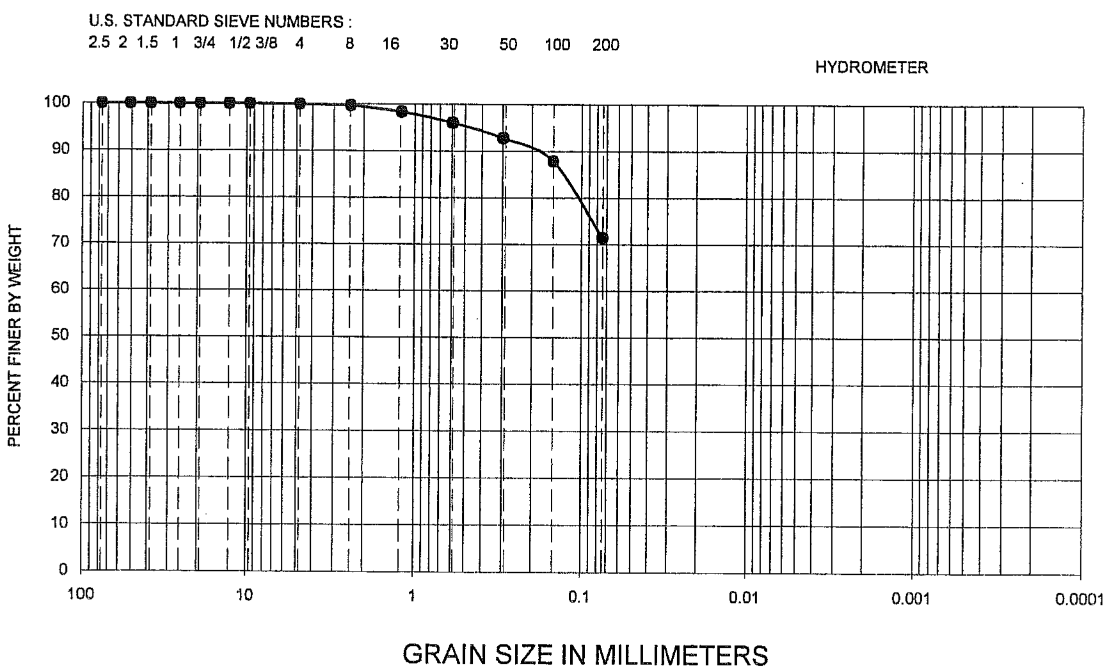
CLIENT:	Celebrate Homes	LAB NO.:	1903
PROJECT:	Seal Beach	PROJECT: NO.:	2888-SD3
MATERIAL LOCATION:	B4 @ 25	DATE:	8.31.05

SAMPLE DESCRIPTION

TOTAL WT. SAMPLE (DRY)	298.9 Dry	**WT. COARSE (+) # 4**	0.2	Dry	**WT COARSE %**	0.1
Wet Wt. Before Wash (-)#4	298.7 Wet	**WT. FINE (-) # 4**	298.7	Wet	**WT FINE %**	99.9
Dry Wt. Before Wash (-)#4	298.7 Dry		298.7	Dry	**-200%**	71
			0	Moisture Content (- # 4)		0

Sieve Size	WEIGHT RETAINED Ind	WEIGHT RETAINED Cum		% RETAINED Ind	% RETAINED Cum		Combined % Passing	Specs.
3"/75mm		0			0		100	
2"/50mm		0			0		100	
1.5"/37.5mm		0			0		100	
1"/25mm		0			0		100	
.75"/19mm		0			0		100	
.5"/12.5mm		0			0		100	
.375"/9.5mm		0			0		100	
#4/4.75mm		0.2			0		100	
#8		0.9		()	(100)		100	
#16		4.8		(2)	(98)		98	
#30		11.7		(4)	(96)		96	
#50		21.4		(7)	(93)		93	
#100		36		(12)	(88)		88	
#200		85.2		(29)	(71)		71	
PAN		92.6					0	
WASH		213.5						

Notes:
all weights are in grams



SIEVE ANALYSIS of COARSE & FINE AGGREGATE

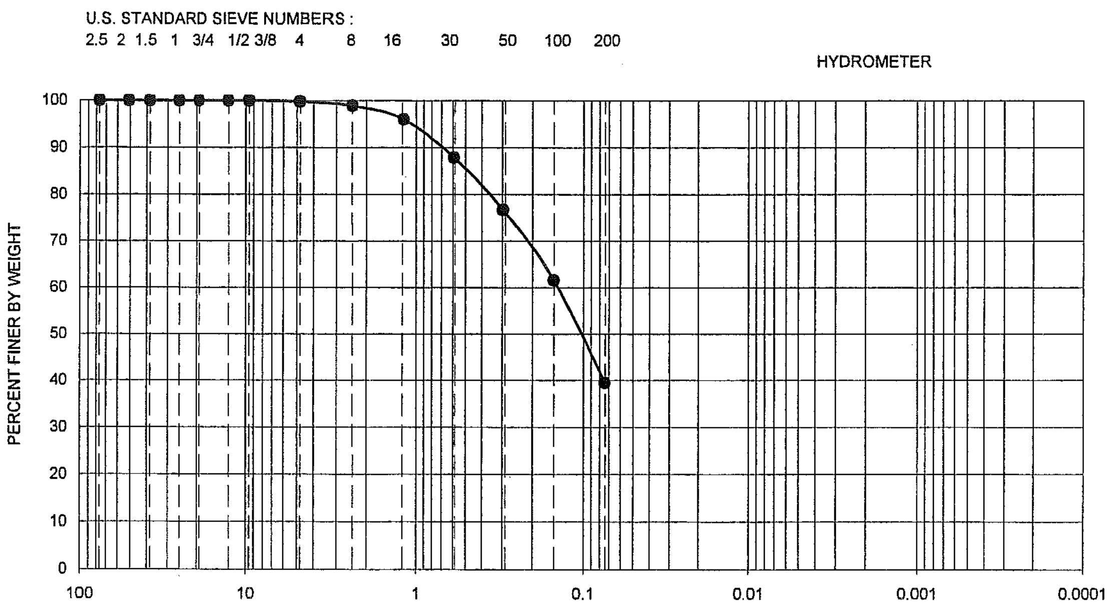
CLIENT:	Celebrate Homes	LAB NO.:	1903
PROJECT:	Seal Beach	PROJECT: NO.:	2888-SD3
MATERIAL LOCATION:	B4 @ 30	DATE:	8.31.05

SAMPLE DESCRIPTION

TOTAL WT. SAMPLE (DRY)	424.4 Dry	**WT. COARSE (+) # 4**	1	Dry	**WT COARSE %**	0.2
Wet Wt. Before Wash (-)#4	423.4 Wet	**WT. FINE (-) # 4**	423.4	Wet	**WT FINE %**	99.8
Dry Wt. Before Wash (-)#4	423.4 Dry		423.4	Dry	**-200%**	40
			0	Moisture Content (- # 4)		0

Sieve Size	WEIGHT RETAINED Ind	WEIGHT RETAINED Cum	% RETAINED	% RETAINED Ind	% RETAINED Cum		Combined % Passing	Specs.
3"/75mm		0			0		100	
2"/50mm		0			0		100	
1.5"/37.5mm		0			0		100	
1"/25mm		0			0		100	
.75"/19mm		0			0		100	
.5"/12.5mm		0			0		100	
.375"/9.5mm		0			0		100	
#4/4.75mm		1			0		100	
#8		3.6		(1)	(99)		99	
#16		15.8		(4)	(96)		96	
#30		50.6		(12)	(88)		88	
#50		98		(23)	(77)		77	
#100		162.1		(38)	(62)		62	
#200		255.7		(60)	(40)		40	
PAN		262.4					0	
WASH		167.7						

Notes:
all weights are in grams



GRAIN SIZE IN MILLIMETERS

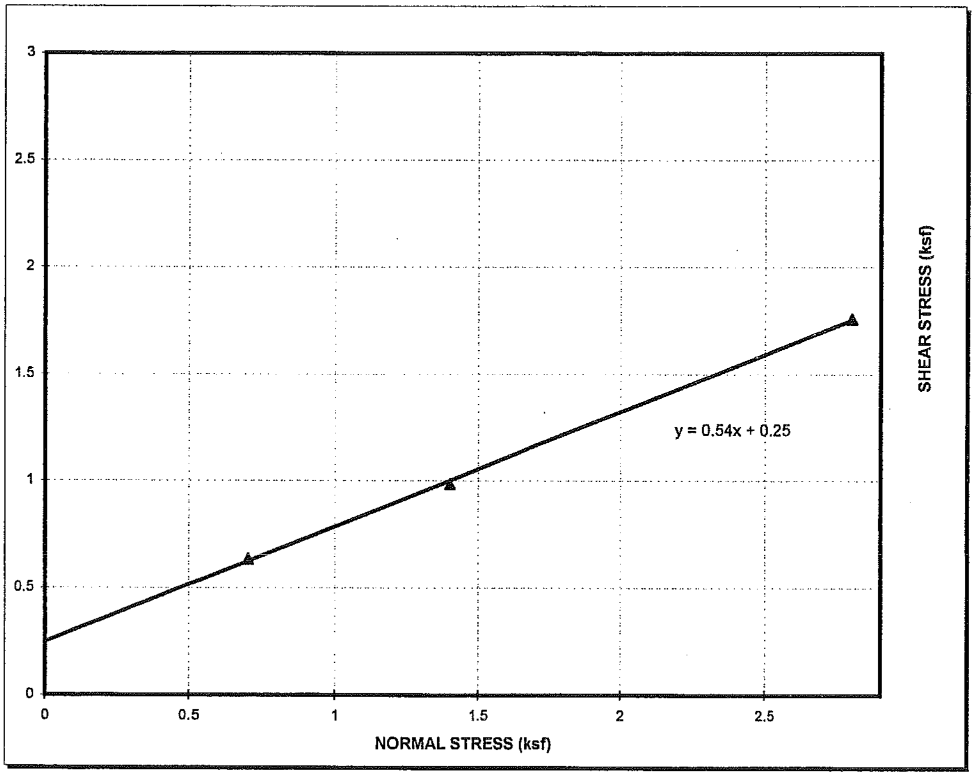
GEO TEK, INC.

DIRECT SHEAR TEST

Project Name: Seal Beach
Project Number: 2888 SD3

Sample Source: B2 @ 0-2'
Date Tested: 08/27/05

Soil Description: Dark Yellowish Brown Silty Fine to Medium Coarse Sand



Shear Strength: $\Phi =$ 28.4 ^{o} , C = 0.25 ksf

Test No.	Load (ksf)	Water Content (%)	Dry Density (pcf)
1	0.7	10	115.8
2	1.4	10	115.6
3	2.8	10	115.6

Note: Saturated in shear box

Notes:
1 - The soil specimen used in the shear box were "ring" samples collected during the field investigation.
2 - Shear strength calculated at 5% of load.
3 - The tests were ran at a shear rate of 0.03 in/min.

L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993 Fax 425-7917 Established 1928

C L A R K S O N L A B O R A T O R Y A N D S U P P L Y I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L A N D C O N S U L T I N G C H E M I S T S

Date: August 31, 2005
Purchase Order Number: 2888SD3
Sales Order Number: 80408
Account Number: GEOT

To:

GeoTek,Inc.
1384 Poinsetta Avenue, Suite A
Vista, CA 92083
Attention: David Cliff

Laboratory Number: SO8612 Customers Phone: 760-599-0509
Fax: 760-599-0593

Sample Designation:

One soil sample received on 8/31/05, taken from Celebrate Homes, Seal Beach marked as Lab#1903 B3-1 @ 0-2.

Analysis By California Test 643, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.2

Water Added (ml)	Resistivity (ohm-cm)
50	868
50	327
50	301
50	281
50	274
50	254
50	254
50	274
50	287

17 years to perforation for a 16 gauge metal culvert.
23 years to perforation for a 14 gauge metal culvert.
31 years to perforation for a 12 gauge metal culvert.
40 years to perforation for a 10 gauge metal culvert.
49 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.039%

Jeff Shannon
Jeff Shannon
JS/arr

APPENDIX C

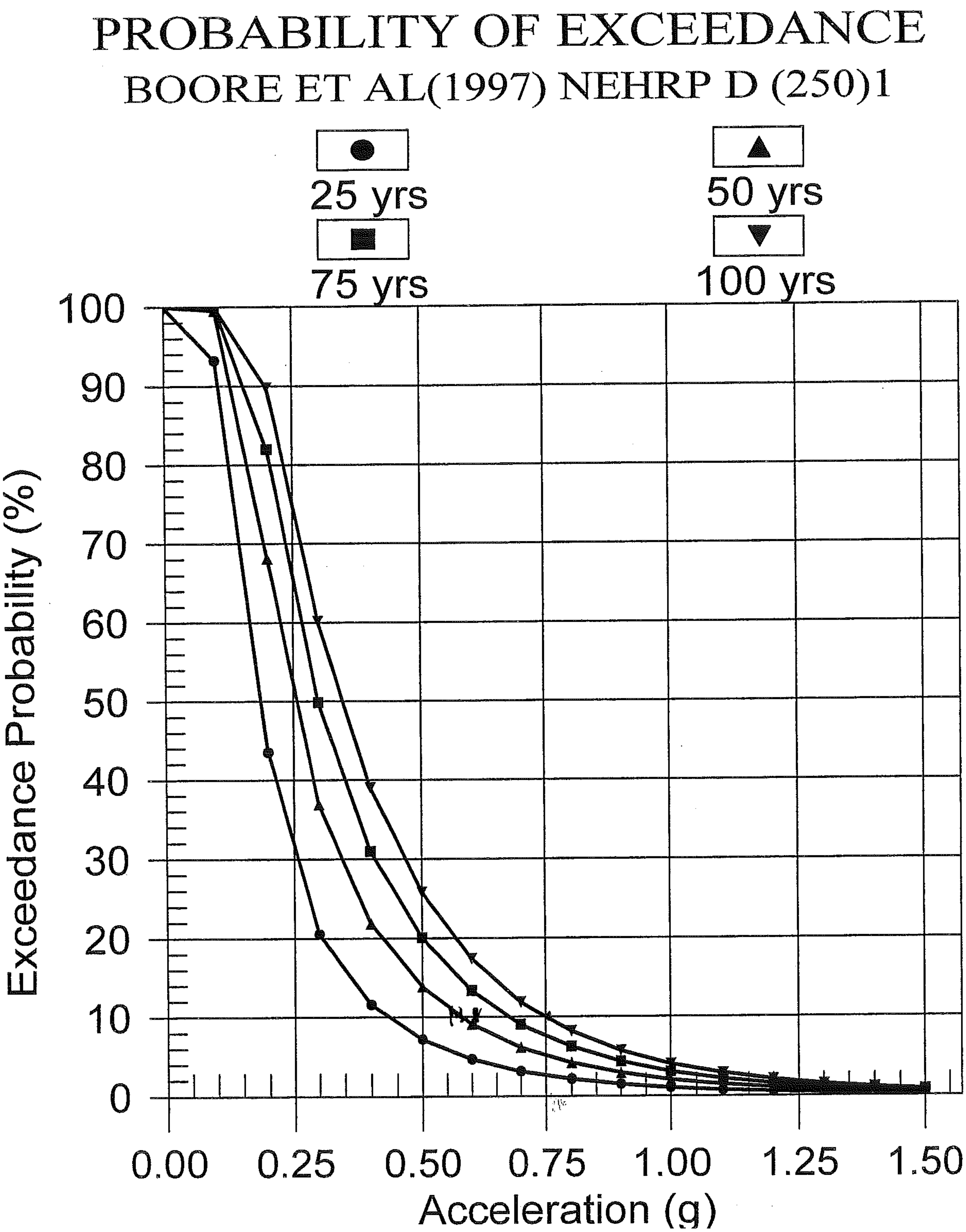
LIQUEFACTION EVALUATION DATA

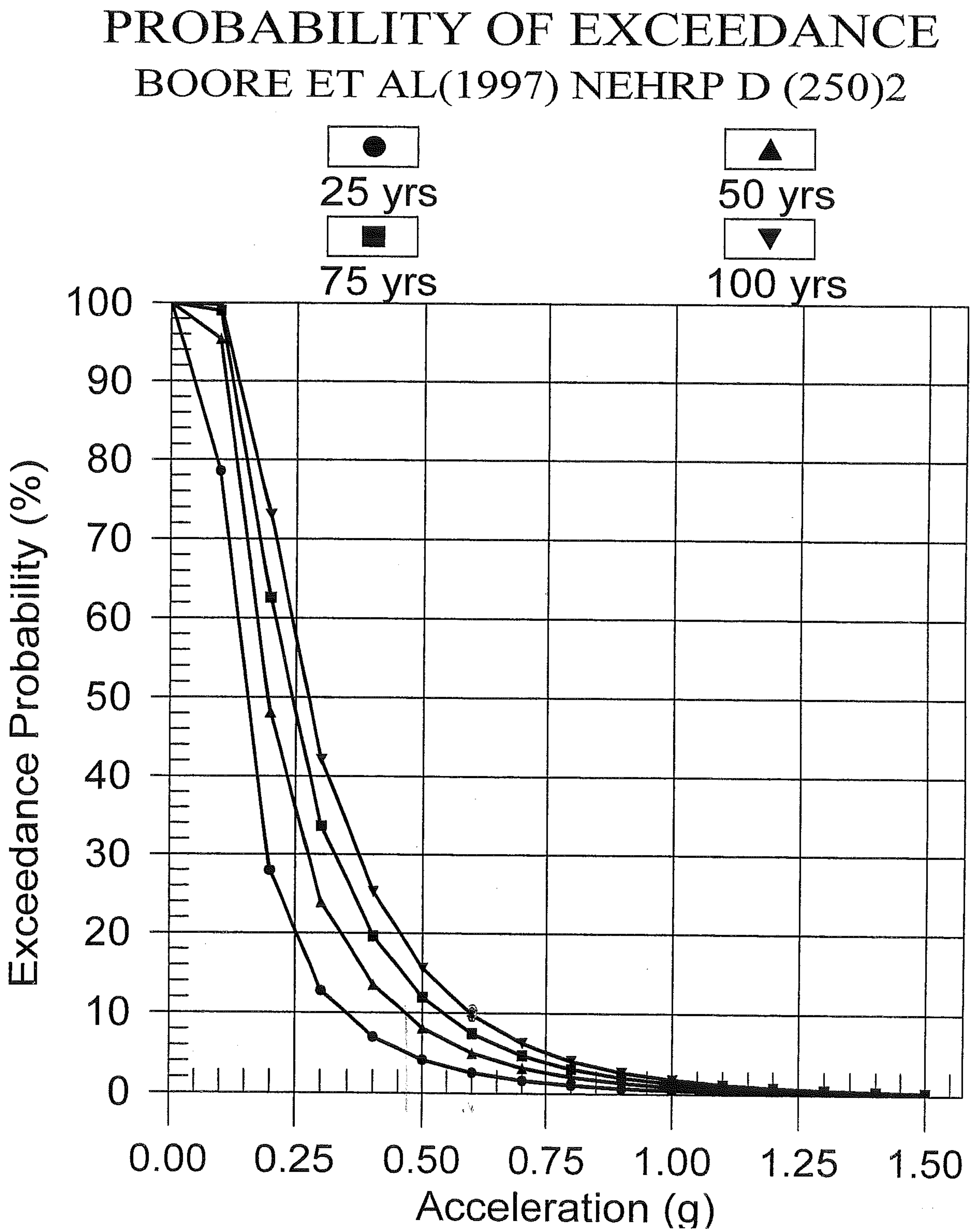
COMPUTER PRINTOUTS OF SEISMIC ANALYSIS

PROPOSED RESIDENTIAL DEVELOPMENT

SEAL BEACH, ORANGE COUNTY, CALIFORNIA

PROJECT NO.: 2888SD3


PROBABILITY OF EXCEEDANCE
BOORE ET AL(1997) NEHRP D (250)1
25 yrs
50 yrs
75 yrs
100 yrs
Exceedance Probability (%)
100
90
80
70
60
50
40
30
20
10
0
0.00
0.25
0.50
0.75
1.00
1.25
1.50
Acceleration (g)


PROBABILITY OF EXCEEDANCE
BOORE ET AL(1997) NEHRP D (250)2
25 yrs
50 yrs
75 yrs
100 yrs
Exceedance Probability (%)
100
90
80
70
60
50
40
30
20
10
0
0.00
0.25
0.50
0.75
1.00
1.25
1.50
Acceleration (g)

GEO TEK, INC.

EVALUATION OF LIQUEFACTION POTENTIAL
DUE TO EARTHQUAKE SHAKING

PROJECT NO.: 2888SD3
PROJECT NAME: Celebrate Homes/Seal Beach
PERFORMED BY: WRM
DATE: 9/13/2005

SOILS/SEISMIC INFORMATION:

WET UNIT WEIGHT ABOVE WATER (P.C.F.): 120
WET UNIT WEIGHT BELOW WATER (P.C.F.): 120
DEPTH TO WATER (FEET): 9
EARTHQUAKE MAGNITUDE: 7.5
MAXIMUM GROUND ACCELERATION (g's): 0.47

BORING NUMBER	DEPTH TO MIDDLE LAYER (FT)	THICKNESS OF LAYER (FEET)	SPT "N" VALUE	FINES CONTENT (%)	TOTAL OVERBURDEN (P.S.F.)	EFFECTIVE OVERBURDEN (P.S.F.)	(N1-60)cs FACTOR	CYCLIC STRESS RATIO IN SOIL	CYCLIC RESISTANCE RATIO	FACTOR OF SAFETY
B-4	10.50	3.00	10	5	1260	1166.4	13.3	0.32	0.15	0.45
B-4	13.25	2.50	4	67	1590	1324.8	10.9	0.35	0.11	0.31
B-4	15.75	2.50	6	67	1890	1468.8	13.4	0.38	0.15	0.38
B-4	18.25	2.50	7	67	2190	1612.8	14.4	0.40	0.16	0.39
B-4	21.25	3.50	9	67	2550	1785.6	16.6	0.41	0.18	0.43
B-4	24.50	3.00	8	71	2940	1972.8	14.8	0.43	0.16	0.37
B-4	29.50	7.00	12	12	3540	2260.8	13.3	0.44	0.15	0.33
B-4	35.50	5.00	51	12	4260	2606.4	47.4	0.44	5.00	>5.00
B-4	40.50	5.00	18	67	4860	2894.4	22.9	0.43	0.25	0.57
B-4	46.25	6.50	41	12	5550	3225.6	43.4	0.42	5.00	>5.00
B-4	50.50	2.00	58	5	6060	3470.4	43.5	0.40	5.00	>5.00

References:
(1) Seed, H. B. and Idriss, I. M., 1982, "Ground Motion and Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute Monograph.
(2) Tokimatsu, K. and Seed, H. B., 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of Geotech. Eng. Division, ASCE, Vol. 113, No. 8, August.
(3) Youd, T.L. and Idriss, I.M., "Proceedings of the NCEER Worshop on Evaluation of Liquefaction Resistance of Soils", Technical Report NCEER-97-0022, December 31, 1997.
(4) SC/EC, 1999, "Recommended Procedures for Implementation of DMG Special Publication 117," Guidelines for Analyzing and Mitigating iquefaction in California, USC, March.

GEO TEK, INC.

EVALUATION OF LIQUEFACTION POTENTIAL DUE TO EARTHQUAKE SHAKING

PROJECT NO.: 2888SD3
PROJECT NAME: Celebrate Homes/Seal Beach
PERFORMED BY: WRM
DATE: 9/13/2005

SOILS/SEISMIC INFORMATION:

WET UNIT WEIGHT ABOVE WATER (P.C.F.): 120
WET UNIT WEIGHT BELOW WATER (P.C.F.): 120
DEPTH TO WATER (FEET): 9
EARTHQUAKE MAGNITUDE: 7.5
MAXIMUM GROUND ACCELERATION (g's): 0.47

BORING NUMBER	DEPTH TO MIDDLE LAYER (FT)	THICKNESS OF LAYER (FEET)	SPT "N" VALUE	FINES CONTENT (%)	TOTAL OVERBURDEN (P.S.F.)	EFFECTIVE OVERBURDEN (P.S.F.)	(N1-60)cs FACTOR	CYCLIC STRESS RATIO IN SOIL	CYCLIC RESISTANCE RATIO	FACTOR OF SAFETY
B-1	11.00	2.00	18	5	1320	1195.2	23.9	0.33	0.26	0.80
B-1	13.25	2.50	29	5	1590	1324.8	35.4	0.35	5.00	>5.00
B-1	16.50	4.00	45	13	1980	1512	55.1	0.39	5.00	>5.00
B-1	20.75	4.50	33	13	2490	1756.8	38.8	0.41	5.00	>5.00
B-1	25.50	5.00	50	13	3060	2030.4	53.7	0.43	5.00	>5.00
B-1	30.75	5.50	30	13	3690	2332.8	30.5	0.44	0.60	1.36
B-1	35.75	4.50	35	12	4290	2620.8	33.0	0.44	5.00	>5.00
B-1	40.50	5.00	104	12	4860	2894.4	90.6	0.43	5.00	>5.00
B-1	47.25	8.50	15	12	5670	3283.2	13.6	0.41	0.15	0.35
B-1	53.75	4.50	30	5	6450	3657.6	21.9	0.41	0.23	0.57
B-1	57.50	3.00	46	5	6900	3873.6	32.7	0.38	5.00	>5.00

References:

(1) Seed, H. B. and Idriss, I. M., 1982, "Ground Motion and Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute Monograph.
(2) Tokimatsu, K. and Seed, H. B., 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of Geotech. Eng. Division, ASCE, Vol. 113, No. 8, August.
(3) Youd, T.L. and Idriss, I.M., "Proceedings of the NCEER Worshop on Evaluation of Liquefaction Resistance of Soils", Technical Report NCEER-97-0022, December 31, 1997.
(4) SC/EC, 1999, "Recommended Procedures for Implementation of DMG Special Publication 117," Guidelines for Analyzing and Mitigating iquefaction in California, USC, March.

GEO TEK, INC.

ESTIMATION OF SEISMICALLY INDUCED SETTLEMENT

PROJECT NO.: 2888SD3
PROJECT NAME: Celebrate Homes/Seal Beach
PERFORMED BY: WRM
DATE: 9/13/2005

SOILS/SEISMIC INFORMATION:

WET UNIT WEIGHT ABOVE WATER (P.C.F.): 120
WET UNIT WEIGHT BELOW WATER (P.C.F.): 120
DEPTH TO WATER (FEET): 9
EARTHQUAKE MAGNITUDE: 7.5
MAXIMUM GROUND ACCELERATION (g's): 0.35

BORING NUMBER	DEPTH TO MIDDLE LAYER (FT)	THICKNESS OF LAYER (FEET)	SPT "N" VALUE	FINES CONTENT (%)	TOTAL OVERBURDEN (P.S.F.)	EFFECTIVE OVERBURDEN (P.S.F.)	(N1-60)cs FACTOR	CYCLIC STRESS RATIO IN SOIL	VOLUMETRIC STRAIN (PERCENT)	ESTIMATED SETTLEMENT (INCHES)
B-4	10.50	3.00	10	5	1260	1166.4	13.3	0.32	2.05	0.74
B-4	13.25	2.50	4	67	1590	1324.8	9.4	0.35	2.70	0.81
B-4	15.75	2.50	6	67	1890	1468.8	11.5	0.38	2.30	0.69
B-4	18.25	2.50	7	67	2190	1612.8	12.3	0.40	2.20	0.66
B-4	21.25	3.50	9	67	2550	1785.6	14.2	0.41	2.00	0.84
B-4	24.50	3.00	8	71	2940	1972.8	12.7	0.43	2.20	0.79
B-4	29.50	7.00	12	12	3540	2260.8	14.4	0.44	2.00	1.68
B-4	35.50	5.00	51	12	4260	2606.4	45.4	0.44	0.00	0.00
B-4	40.50	5.00	18	67	4860	2894.4	19.4	0.43	1.60	0.96
B-4	46.25	6.50	41	12	5550	3225.6	33.0	0.42	0.00	0.00
B-4	50.50	2.00	58	5	6060	3470.4	43.5	0.40	0.00	0.00

TOTAL ESTIMATED SETTLEMENT = 7.17 INCHES

References:
(1) Seed, H. B. and Idriss, I. M., 1982, "Ground Motion and Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute Monograph.
(2) Tokimatsu, K. and Seed, H. B., 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of Geotech. Eng. Division, ASCE, Vol. 113, No. 8, August.
(3) Youd, T.L. and Idriss, I.M., "Proceedings of the NCEER Worshop on Evaluation of Liquefaction Resistance of Soils", Technical Report NCEER-97-0022, December 31, 1997.
(4) SC/EC, 1999, "Recommended Procedures for Implementation of DMG Special Publication 117," Guidelines for Analyzing and Mitigating iquefaction in California, USC, March.

GEO TEK, INC.

ESTIMATION OF SEISMICALLY INDUCED SETTLEMENT

PROJECT NO.: 2888SD3
PROJECT NAME: Celebrate Homes/Seal Beach
PERFORMED BY: WRM
DATE: 9/13/2005

SOILS/SEISMIC INFORMATION:

WET UNIT WEIGHT ABOVE WATER (P.C.F.): 120
WET UNIT WEIGHT BELOW WATER (P.C.F.): 120
DEPTH TO WATER (FEET): 9
EARTHQUAKE MAGNITUDE: 7.5
MAXIMUM GROUND ACCELERATION (g's): 0.35

BORING NUMBER	DEPTH TO MIDDLE LAYER (FT)	THICKNESS OF LAYER (FEET)	SPT "N" VALUE	FINES CONTENT (%)	TOTAL OVERBURDEN (P.S.F.)	EFFECTIVE OVERBURDEN (P.S.F.)	(N1-60)cs FACTOR	CYCLIC STRESS RATIO IN SOIL	VOLUMETRIC STRAIN (PERCENT)	ESTIMATED SETTLEMENT (INCHES)
B-1	11.00	2.00	18	5	1320	1195.2	23.9	0.33	1.00	0.24
B-1	13.25	2.50	29	5	1590	1324.8	35.4	0.35	0.00	0.00
B-1	16.50	4.00	45	13	1980	1512	52.3	0.39	0.00	0.00
B-1	20.75	4.50	33	13	2490	1756.8	36.6	0.41	0.00	0.00
B-1	25.50	5.00	50	13	3060	2030.4	51.0	0.43	0.00	0.00
B-1	30.75	5.50	30	13	3690	2332.8	28.6	0.44	0.75	0.50
B-1	35.75	4.50	35	12	4290	2620.8	31.5	0.44	0.50	0.27
B-1	40.50	5.00	104	12	4860	2894.4	87.3	0.43	0.00	0.00
B-1	47.25	8.50	15	12	5670	3283.2	12.7	0.41	2.20	2.24
B-1	53.75	4.50	30	5	6450	3657.6	21.9	0.41	1.40	0.76
B-1	57.50	3.00	46	5	6900	3873.6	32.7	0.38	0.00	0.00

TOTAL ESTIMATED SETTLEMENT = 4.01 INCHES

References:
(1) Seed, H. B. and Idriss, I. M., 1982, "Ground Motion and Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute Monograph.
(2) Tokimatsu, K. and Seed, H. B., 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of Geotech. Eng. Division, ASCE, Vol. 113, No. 8, August.
(3) Youd, T.L. and Idriss, I.M., "Proceedings of the NCEER Worshop on Evaluation of Liquefaction Resistance of Soils", Technical Report NCEER-97-0022, December 31, 1997.
(4) SC/EC, 1999, "Recommended Procedures for Implementation of DMG Special Publication 117," Guidelines for Analyzing and Mitigating iquefaction in California, USC, March.

APPENDIX D

GENERAL GRADING GUIDELINES
FOR EARTHWORK CONSTRUCTION

PROPOSED RESIDENTIAL DEVELOPMENT
SEAL BEACH, ORANGE COUNTY, CALIFORNIA
PROJECT NO.: 2888SD3

GRADING GUIDELINES

Guidelines presented herein are intended to address general construction procedures for earthwork construction. Specific situations and conditions often arise which cannot reasonably be discussed in general guidelines, when anticipated these are discussed in the text of the report. Often unanticipated conditions are encountered which may necessitate modification or changes to these guidelines. It is our hope that these will assist the contractor to more efficiently complete the project by providing a reasonable understanding of the procedures that would be expected during earthwork and the testing and observation used to evaluate those procedures.

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the Uniform Building Code and the guidelines presented below.

Preconstruction Meeting

A preconstruction meeting should be held prior to site earthwork. Any questions the contractor has regarding our recommendations, general site conditions, apparent discrepancies between reported and actual conditions and/or differences in procedures the contractor intends to use should be brought up at that meeting. The contractor (including the main onsite representative) should review our report and these guidelines in advance of the meeting. Any comments the contractor may have regarding these guidelines should be brought up at that meeting.

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The Contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations, our observations and test results are intended to evaluate the contractor's overall level of efforts during grading. The contractor's personnel are the only individuals participating in all aspect of site work. Compaction

testing and observation should not be considered as relieving the contractor's responsibility to properly compact the fill.

3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the Contractor's responsibility to notify our representative or office when such areas are ready for observation.
4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.
5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.
6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
   a) Density tests should be taken periodically during grading on the flat surface of the fill three to five feet horizontally from the face of the slope.
   b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

1. All vegetation, and other deleterious materials, should be removed from the site. If material is not immediately removed from the site it should be stockpiled in a designated area(s) well outside of all current work areas and delineated with flagging or other means. Site clearing should be performed in advance of any grading in a specific area.

2. Efforts should be made by the contractor to remove all organic or other deleterious material from the fill, as even the most diligent efforts may result in the incorporation of some materials. This is especially important when grading is occurring near the natural grade. All equipment operators should be aware of these efforts. Laborers may be required as root pickers.
3. Nonorganic debris or concrete may be placed in deeper fill areas provided the procedures used are observed and found acceptable by our representative. Typical procedures are similar to those indicated on Plate G-4.

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium and colluvium as well as weathered or creep effected bedrock, should be removed (see Plates G-1, G-2 and G-3) unless otherwise specifically indicated in the text of this report.
2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient) the contractor should not exceed these depths unless directed otherwise by our representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.
5. Exploratory back hoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Subdrainage

1. Subdrainage systems should be provided in canyon bottoms prior to placing fill, and behind buttress and stabilization fills and in other areas indicated in the report. Subdrains should conform to schematic diagrams G-1 and G-5, and be acceptable to our representative.
2. For canyon subdrains, runs less than 500 feet may use six-inch pipe. Typically, runs in excess of 500 feet should have the lower end as eight-inch minimum.
3. Filter material should be clean, 1/2 to 1-inch gravel wrapped in a suitable filter fabric. Class 2 permeable filter material per California Department of Transportation Standards tested by this office to verify its suitability, may be used without filter fabric. A sample of the material should be provided to the Soils Engineer by the contractor at least two working days before it is delivered to the site. The filter should be clean with a wide range of sizes.

4. Approximate delineation of anticipated subdrain locations may be offered at 40-scale plan review stage. During grading, this office would evaluate the necessity of placing additional drains.
5. All subdrainage systems should be observed by our representative during construction and prior to covering with compacted fill.
6. Subdrains should outlet into storm drains where possible. Outlets should be located and protected. The need for backflow preventers should be assessed during construction.
7. Consideration should be given to having subdrains located by the project surveyors.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm , the Contractor should rework the fill until it is in accordance with the following:
    a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
    b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D-1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
    a) They are not placed in concentrated pockets;
    b) There is a sufficient percentage of fine-grained material to surround the rocks;

c) The distribution of the rocks is observed by and acceptable to our representative.

5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal (See Plate G-4). On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.
6. In clay soil dry or large chunks or blocks are common; if in excess of eight (8) inches minimum dimension then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

1. The Contractor should obtain a minimum relative compaction of 90 percent out to the finished slope face of fill slopes. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment.
2. Slopes trimmed to the compacted core should be overbuilt by at least three (3) feet with compaction efforts out to the edge of the false slope. Failure to properly compact the outer edge results in trimming not exposing the compacted core and additional compaction after trimming may be necessary.
3. If fill slopes are built "at grade" using direct compaction methods then the slope construction should be performed so that a constant gradient is maintained throughout construction. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Compaction equipment should compact each lift along the immediate top of slope. Slopes should be back rolled or otherwise compacted at approximately every 4 feet vertically as the slope is built.
4. Corners and bends in slopes should have special attention during construction as these are the most difficult areas to obtain proper compaction.
5. Cut slopes should be cut to the finished surface, excessive undercutting and smoothing of the face with fill may necessitate stabilization.

Keyways, Buttress and Stabilization Fills

Keyways are needed to provide support for fill slope and various corrective procedures.

1. Side-hill fills should have an equipment-width key at their toe excavated through all surficial soil and into competent material and tilted back into the hill (Plates G-2, G-3). As the fill is elevated, it should be benched through surficial soil and slopewash, and into competent bedrock or other material deemed suitable by our representatives (See Plates G-1, G-2, and G-3).
2. Fill over cut slopes should be constructed in the following manner:
   a) All surficial soils and weathered rock materials should be removed at the cut-fill interface.
   b) A key at least one (1) equipment width wide (or as needed for compaction) and tipped at least one (1) foot into slope should be excavated into competent materials and observed by our representative.
   c) The cut portion of the slope should be excavated prior to fill placement to evaluate if stabilization is necessary, the contractor should be responsible for any additional earthwork created by placing fill prior to cut excavation.

   See Plate G-3 for schematic details.

3. Daylight cut lots above descending natural slopes may require removal and replacement of the outer portion of the lot. A schematic diagram for this condition is presented on Plate G-2.
4. A basal key is needed for fill slopes extending over natural slopes. A schematic diagram for this condition is presented on Plate G-2.
5. All fill slopes should be provided with a key unless within the body of a larger overall fill mass. Please refer to Plate G-3, for specific guidelines.

Anticipated buttress and stabilization fills are discussed in the text of the report. The need to stabilize other proposed cut slopes will be evaluated during construction. Plate G-5 is shows a schematic of buttress construction.

1. All backcuts should be excavated at gradients of 1:1 or flatter. The backcut configuration should be determined based on the design, exposed conditions and need to maintain a minimum fill width and provide working room for the equipment.
2. On longer slopes backcuts and keyways should be excavated in maximum 250 feet long segment. The specific configurations will be determined during construction.
3. All keys should be a minimum of two (2) feet deep at the toe and slope toward the heel at least one foot or two (2%) percent whichever is greater.

4. Subdrains are to be placed for all stabilization slopes exceeding 10 feet in height. Lower slopes are subject to review. Drains may be required. Guidelines for subdrains are presented on Plate G-5.
5. Benching of backcuts during fill placement is required.

Lot Capping

1. When practical, the upper three (3) feet of material placed below finish grade should be comprised of the least expansive material available. Preferably, highly and very highly expansive materials should not be used. We will attempt to offer advise based on visual evaluations of the materials during grading, but it must be realized that laboratory testing is needed to evaluate the expansive potential of soil. Minimally, this testing takes two (2) to four (4) days to complete.
2. Transition lots (cut and fill) both per plan and those created by remedial grading (e.g. lots above stabilization fills, along daylight lines, above natural slope, etc.) should be capped with a three foot thick compacted fill blanket.
3. Cut pads should be observed by our representative(s) to evaluate the need for overexcavation and replacement with fill. This may be necessary to reduce water infiltration into highly fractured bedrock or other permeable zones, and/or due to differing expansive potential of materials beneath a structure. The overexcavation should be at least three feet. Deeper overexcavation may be recommended in some cases.

OVERSIZED ROCK PLACEMENT

Oversize material could be generated during grading. Such materials may require special handling for burial. Although alternatives may be developed in the field, the following methods of rock disposal are recommended on a preliminary basis.

Limited Larger Rock

When materials encountered are principally soil with limited quantities of larger rock fragments or boulders, placement in windrows is recommended. The following procedures should be applied:

1. Oversize rock (greater than 8 inch) should be placed in windrows.
   a) Windrows are rows of single file rocks placed to avoid nesting or clusters of rock.

b) Each adjacent rock should be approximately the same size (within ~one foot in diameter).

c) The maximum rock size allowed in windrows is four feet

2. A minimum vertical distance of three feet between lifts should be maintained. Also, the windrows should be offset from lift to lift. Rock windrows should not be closer than 15 feet to the face of fill slopes and sufficient space must be maintained for proper slope construction (see Plate G-4).
3. Rocks greater than eight inches in diameter should not be placed within seven feet of the finished subgrade for a roadway or pads and should be held below the depth of the lowest utility. This will allow easier trenching for utility lines.
4. Rocks greater than four feet in diameter should be broken down, if possible, or they may be placed in a dozer trench. Each trench should be excavated into the compacted fill a minimum of one foot deeper than the largest diameter of rock.
   a) The rock should be placed in the trench and granular fill materials (SE>30) should be flooded into the trench to fill voids around the rock.
   b) The over size rock trenches should be no closer together than 15 feet from any slope face.
   c) Trenches at higher elevation should be staggered and there should be a minimum of four feet of compacted fill between the top of the one trench and the bottom of the next higher trench.
   d) It would be necessary to verify 90 percent relative compaction in these pits. A 24 to 72 hour delay to allow for water dissipation should be anticipated prior to additional fill placement.

UTILITY TRENCH CONSTRUCTION AND BACKFILL

Utility trench excavation and backfill is the contractors responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While, efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.

Compaction methods vary for trench compaction and experience indicates many methods can be successful. However, procedures that "worked" on previous projects may or may not prove effective on a given site. The contractor(s) should outline the procedures proposed, so that we may discuss them **prior** to construction. We will offer comments based on our knowledge of site conditions and experience.

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing the trench.
2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
   a) shallow (12 + inches) under slab interior trenches and,
   b) as bedding in pipe zone.

   The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.
3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.
4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractors procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractors attention.

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries our safety considerations for use by all our employees on multi-employer construction sites. On ground personnel are at highest risk of injury and possible fatality on grading construction projects. The company recognizes that construction activities will vary on each site and that job site safety is the contractor's responsibility. However, it is, imperative that all personnel be safety conscious to avoid accidents and potential injury.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of our field personnel on grading and construction projects.

1. Safety Meetings: Our field personnel are directed to attend the contractor's regularly scheduled safety meetings.

2. Safety Vests: Safety vests are provided for and are to be worn by our personnel while on the job site.

3. Safety Flags: Safety flags are provided to our field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

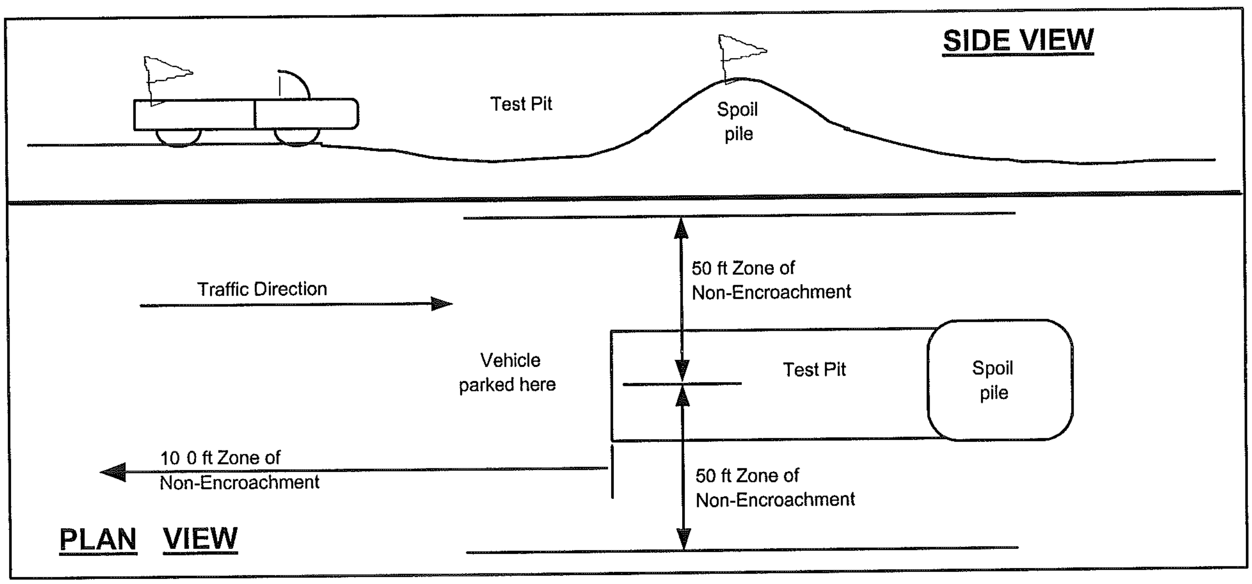
Test Pits Location, Orientation and Clearance

The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading contractors authorized representatives (e.g. dump man, operator, supervisor, grade checker, etc.), and to select locations following or behind the established traffic pattern, preferable outside of current traffic. The contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates that the fill be maintained in a drivable condition. Alternatively, the contractor may opt to park a piece of equipment in front of test pits, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits (see diagram below) No grading equipment should enter this zone during the test procedure. The zone should extend outward to the sides approximately 50 feet from the center of the test pit and 100 feet in the direction of traffic flow. This zone is established both for safety and to avoid excessive ground vibration, which typically decreases test results.

TEST PIT SAFETY PLAN



Slope Tests

When taking slope tests, the technician should park their vehicle directly above or below the test location on the slope. The contractor's representative should effectively keep all equipment at a safe operation distance (e.g. 50 feet) away from the slope during testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location.

Trench Safety:

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Trenches for all utilities should be excavated in accordance with CAL-OSHA and any other applicable safety standards. Safe conditions will be required to enable compaction testing of the trench backfill.

All utility trench excavations in excess of 5 feet deep, which a person enters, are to be shored or laid back. Trench access should be provided in accordance with OSHA standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

Our personnel are directed not to enter any excavation which;

1. is 5 feet or deeper unless shored or laid back,
2. exit points or ladders are not provide,
3. displays any evidence of instability, has any loose rock or other debris which could fall into the trench, or
4. displays any other evidence of any unsafe conditions regardless of depth.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractors representative will then be contacted in an effort to effect a solution. All backfill not tested due to safety concerns or other reasons is subject to reprocessing and/or removal.

<u>Procedures</u>

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor's representative will then be contacted in an effort to effect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim can be considered unacceptable and subject to reprocessing, recompaction or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technicians attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.